



East Bay Replacement



SAN FRANCISCO-OAKLAND BAY BRIDGE EAST SPAN REPLACEMENT PROJECT

**INITIAL SUBMITTAL TO:
THE METROPOLITAN TRANSPORTATION COMMISSION
ENGINEERING AND DESIGN ADVISORY PANEL**

MAY 12, 1997

CONCEPTS FOR THE REPLACEMENT STRUCTURE

**PARSONS BRINCKERHOFF / HNTB CORPORATION
*a JOINT VENTURE***

in Association with
**SC SOLUTIONS, INC.
DIGITAL STRUCTURES, INC.**



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PROJECT DESCRIPTION

Replacement of the San Francisco - Oakland Bay Bridge East Bay Spans presents a unique and unprecedented challenge to bridge engineering. Just as the design and construction of the existing structure set new standards for the bridge industry in the first half of this century, so too must the new design and construction now redefine performance of major civil engineering structures.

The project consists of three distinct parts for building the new structure plus demolition (or deconstruction) of the existing structure.

1. Transition from Yerba Buena Island Tunnel to the Main Spans

2. Main Spans (or Navigation Spans)

3. Main Spans to the Terminus in Oakland - the Approach Structures

The structure types selected for these portions of the structure will have to consider the relationships of the other two including alignment and profile and will have to be in harmony with the magnificent west bay suspension spans.

Design to maintain serviceability following the maximum credible earthquake requires that the structure type selected have an inherently high level of seismic performance. For this reason the studies undertaken by the PB/HNTB team have focused on cable supported structures. Since this location is impractical for construction of suspension bridges, the focus has been primarily on Cable Stayed structures. This study presents structures considered for both the Main (Navigation) Spans and the Approach spans.

KEY ISSUES

The importance of successfully constructing a bridge to meet the project criteria cannot be underestimated. To meet the needs of transportation in the Bay Area both now and in the future the design of the structural and operational system and its performance requirements must be considered. These include:

Seismic Performance

Cost

Service and Windload Performance

Aesthetics

Traffic management During Construction

Transition Structures to the existing facilities

Alignment and Grade for Optimal Traffic Flow

Flexibility for Future Betterments

Environmental Impacts

Maintenance

Speed of Construction

Our studies to date have included consideration of all the above issues plus many more. But the issue that stands out above all others and clearly is the driving force of the entire project is the **seismic performance** of the bridge

MAIN (NAVIGATION) SPANS

ALTERNATIVE 1 - SINGLE TOWER - TWO SPAN CABLE STAY

ALTERNATIVE 2 - SINGLE DIAMOND TOWER - TWO SPAN CABLE STAY

ALTERNATIVE 3 - DOUBLE DIAMOND TOWER 3-SPAN CABLE STAY

ALTERNATIVE 4 - CABLE STAYED VIADUCT

ALTERNATIVE 5 - SINGLE CELL CIP SEGMENTAL BOX GIRDER

FOUNDATION CONSIDERATIONS

Foundations for the West end of the main span (single tower of the asymmetric cable-stayed alternative, West Tower of the symmetric cable-stay bridge, or Main Span for the segmental box girder) will be founded on the rock at YBI (near E-2), in shallow water. Either drilledshafts filled with reinforced concrete, or a reinforced concrete pier embedded or anchored into the weathered rock will be the preferred foundation for this location. The proposed caissons will be 10' diameter.

The East Tower of the symmetric cable-stay bridge, and the piers for the Viaduct girders will all be located on deep alluvial deposits (soft bay mud, overlaying soft to stiff clays and silts) in water depths up to 40'. Large-diameter driven steel thick-walled pipe piles are the preferred foundation type for these piers due to the relative simplicity of installation and the efficiency of the steel section. Using large pile driving hammers, typically used for marine offshore pile installation, 10'-diameter pipe piles are proposed. The cost-benefit of alternative designs with more 8' or 6'-diameter piles compared to fewer larger piles can be further investigated during preliminary design.

ALTERNATIVES

TWO SPAN ASYMMETRIC CABLE STAYED BRIDGES

(Alternatives 1, 2, & 2A)

The opportunity for shallow foundations in competent rock on and near Yerba Buena Island (YBI) together with the proximity of the navigation channel suggests a single tower cable-stayed bridge of a moderate main-span. A single cross-section carrying 10 lanes of traffic and shoulders is proposed. This unusually wide cross section strongly impacts tower and cable configurations.

The proposed span arrangement consists of a 600 ft side span and a 750 ft main span with an additional 250 ft +/- from the adjacent viaduct span required for balanced cantilever construction.. A longer main span is possible but is not warranted due to the 500 ft navigational clearance requirement. Two intermediate side span piers are provided on Yerba Buena Island to enhance live load and seismic performance as well as constructability.

ALTERNATIVE 1

SINGLE PYLON - ASYMMETRIC CABLE STAY

A single shaft pylon alternative supporting a multi-cell concrete box section is adopted. The box section is comprised of two single cell cast-in-place concrete boxes connected transversely with precast slab panels. Cantilever precast and cast-in-place segments are installed following erection of the boxes. Cables are anchored to blisters at the interior webs of the single cell boxes. High torsional rigidity of the cross-section is necessary since cables, anchored near the center of the superstructure, provide little torsional support. A superstructure depth of 12 feet is required to provide both torsional and flexural resistance, resulting in a relatively heavy cross-section as

compared to Alternative 2. Flexible pylon-to-superstructure connections are provided to decouple seismic response between the two elements.

The pylon section is comprised of 4 tapered octagonal shafts connected by a solid wall section at the cable anchorage locations (anchor head) and coupling beams below this elevation. The use of a solid tower head with cables anchored through the section substantially minimizes cross sectional requirements and therefore pylon inertia. Coupling beams enhance flexibility of the pylon and provide an excellent means of seismic resistance.

ALTERNATIVE 2

SINGLE DIAMOND PYLON - ASYMMETRIC CABLE STAY

For this alternative, an inverted Y shaped tower supports a modified channel cross section. In the main span, three planes of cables support the superstructure along the center and each edge, helping to minimize section depth requirements and therefore inertia. The side spans are supported by two planes of cables along the edges and by an 8 ft deep spline beam along the centerline. In both side and main spans, the edge beams are 5 ft in depth at the fascia, resulting in a slender ribbon-like appearance.

The pylon section is comprised of 4 - 6 ft diameter circular shafts connected by a solid wall section at the cable anchorage locations (vertical leg) and shear walls for the remaining inclined legs. The use of a solid tower head with cables anchored through the section minimizes cross sectional requirements as in alternative 1. Due to the relatively stiff seismic response of the pylon section, flexible connections are provided at the tower - superstructure interface to decouple the response of the two elements.

ALTERNATIVE 2A

DOUBLE DIAMOND PYLON - ASYMMETRIC CABLE STAY

This alternative was investigated for cost comparison only. The structure is similar to Alternative 2 with the exception that the pylon is split into two pylons. No plans or details are presented for this alternative.

ALTERNATIVE 3

DOUBLE DIAMOND TOWER SYMMETRICAL CABLE STAYED BRIDGE

This alternative consists of twin bridges with spans of 183m-427m-183m (600-1400-600ft). The structures twin diamond towers rise prominently above the San Francisco Bay just east of Yerba Buena Island and compliment the existing West Suspended Spans by providing a contrasting structure that adds excitement to the Oakland and San Francisco skyline. Double towers also provide the opportunity to furnish cable stays along each edge of both structures which help define and provide a transition to the viaduct approach structures. Tower height is restricted to the height of the West suspension bridge towers to ensure one bridge does not dominate the other.

Span arrangements were chosen in consideration of cost, aesthetics, and seismic performance. With a main span off 427m (1400 ft), the East Bay Bridge will truly be a landmark structure with a main span length exceeded in North America by only the Alex Fraser Bridge in Canada with a span of 465m (1526ft). The main span permits locating the West Pier just off Yerba Buena Island in competent bedrock while providing sufficient span length to avoid placing the East Pier in the deepest section of the navigable channel. In order to improve the seismic behavior of the structure, it is possible to equilibrate lateral stiffness of the east and west tower foundations.

Diamond towers have been chosen due to their aesthetic appeal and excellent capability to resist seismic and wind loads. The diamond towers result in inclined cables that have the advantage of

increasing the torsional stiffness of the superstructure. For a span of 1400 ft, winds demands can be significant and supplying a deck with high torsional stiffness is important.

ALTERNATIVE 4

SINGLE DIAMOND PYLON - VIADUCT CABLE STAY

A multi-span cable-stayed bridge alternative is considered for the viaduct spans. A three span unit of 750 ft with 375 ft side spans comprise a 3000 ft segment between expansion joints. The use of a single cable-supported superstructure system for the east bay is similar in concept to the multi-span suspension bridges crossing the west bay.

A single pylon shaft rises 150 ft above deck level and supports two planes of cables anchored near the center of a multi-cell concrete box cross-section which supports 10 lanes of traffic. The use of low towers minimizes inertia but results in increased flexural demands in the superstructure and an increase in cable quantity. The use of a 12 foot deep section provides sufficient capacity to resist service load and seismic moments.

In order to improve the behavior of multi-span cable-stayed bridges under vertical loads, additional cables are provided at interior spans which overlap with the cables from adjacent exterior spans.

ALTERNATIVE 5

SINGLE CELL CAST IN PLACE SEGMENTAL CONCRETE BOX GIRDER

A concrete box girder alternative was investigated for the replacement of the navigation span. A single cell box girder was assumed to support each roadway. The use of light weight concrete for the box girder with a silica fume concrete wearing surface was assumed. Pier concrete was assumed to be normal weight concrete.

Construction of the box girders is anticipated to be the cast-in-place balanced cantilever method. With a navigation span of 900 feet, the pier on the Oakland side of the channel would be constructed with a pair of 450-foot cast-in-place concrete box girder cantilevers. The Oakland side cantilever would be made continuous with the concrete box girder approach spans which are assumed to be 550-foot spans, resulting in a 725-foot end span. The pier adjacent to Yerba Buena Island would also be constructed with a pair of 450-foot cantilevers. After closure is made at the mid point of the channel span, the 450-foot cantilever on the island side would be extended 100 feet to provide a 550 end span.

For the structure without bicycle lanes, an 25m (83.5 ft) wide, single-cell, box girder with 6.6m (21.75ft) cantilevers was investigated. The box girder was assumed to vary in depth from 15.2m (50 ft) at the piers to 4.6m (15ft) at the end of the cantilevers. The minimum depth of the box girder will be adjusted to match the depth of Approach spans.

APPROACH STRUCTURES

Although the more conventional methods of construction do not have the inherent seismic performance that cable supported structures do, the necessary level of performance can be obtained at the span lengths considered. This has been confirmed with preliminary local seismic analysis and capacity evaluation of the proposed pier configuration.

The viaduct structure which comprises approximately 2600m (8,000 ft.) of bridge for the east spans is such a major component that a span optimization study carried out to come up with an efficient viaduct solution. Four span lengths were considered for these spans - 60m (200ft), 91m (300ft.),

168m (550ft.) and 244m (800ft.). The optimization study was carried out for these span lengths to determine the most efficient span for a given column height and foundation condition. Both normal weight and light weight concrete were considered. Based on integral frames for the taller columns, meaning an integral connection between the superstructure box girder, the piers are designed to be essentially elastic. Shorter columns in the 91m (300ft.) spans will be isolated from the substructure with groups of bearings.

PIERS

The cylindrical pier shaft can be constructed with well established construction practices using slip forming. This type of construction is common in many structures including bridge piers and grain silos. The 24 ft diameter shaft will be designed to remain elastic under a major seismic event. It will also be designed to be ductile.

The pile configuration follows the circular pattern which is more efficient when transverse and longitudinal stiffness have to vary without changing the footing dimensions. The rotation and the positioning of the piles can always change to reflect the actual conditions at each individual pier.

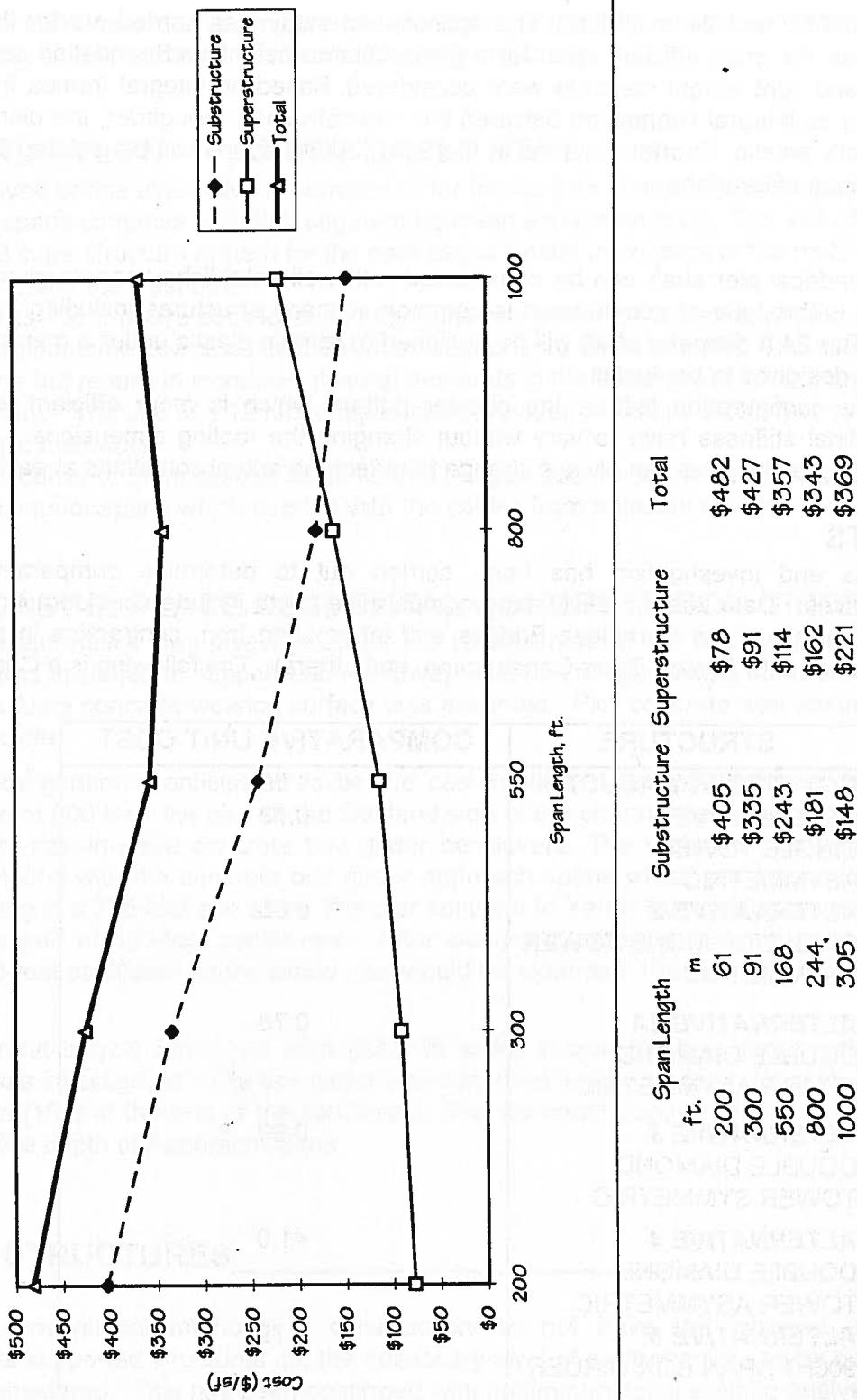
COSTS

Analysis and investigation has been carried out to determine comparative costs of all the alternatives. Data used in developing comparative costs include consideration of unit pricing used for the retrofit of the Carquinez Bridges and information from contractors in the Bay Area (Nesco Fabricators, L.B. Foster, Dutra Construction, and others). The following is a Comparative Cost table.

STRUCTURE	COMPARATIVE UNIT COST
BOX GIRDER VIADUCT	1.00
ALTERNATIVE 1	0.73
SINGLE TOWER ASYMMETRIC	
ALTERNATIVE 2	0.68
SINGLE DIAMOND TOWER ASYMMETRIC	
ALTERNATIVE 2A	0.78
DOUBLE DIAMOND TOWER ASYMMETRIC	
ALTERNATIVE 3	0.69
DOUBLE DIAMOND TOWER SYMMETRIC	
ALTERNATIVE 4	<1.0
DOUBLE DIAMOND TOWER ASYMMETRIC	
ALTERNATIVE 5	0.72
900FT SPAN BOX GIRDER	

**BAY BRIDGE COST ESTIMATE - APPROACH STRUCTURES
NORMAL WEIGHT CONCRETE BOX GIRDERS**

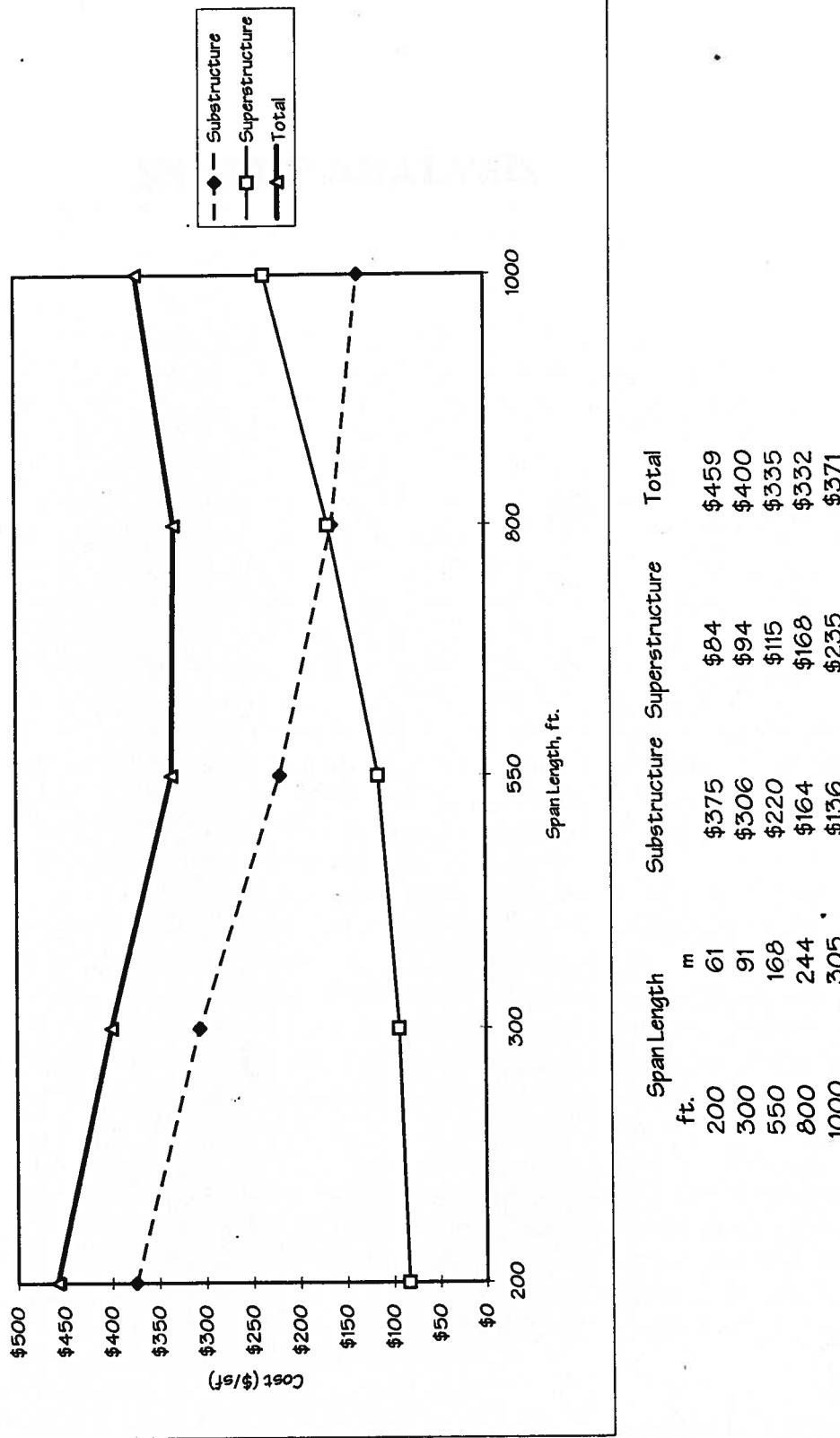
SPAN OPTIMIZATION STUDY



BAY BRIDGE COST ESTIMATE - APPROACH STRUCTURES
LIGHT WEIGHT CONCRETE BOX GIRDERS

PB
100.
Years.

SPAN OPTIMIZATION STUDY



In particular, the nature of the analysis can be illustrated by the following example. In Fig. 10, the bridge is shown in its original configuration, with a central pier, two towers, and two end piers.

SEISMIC ANALYSIS

The bridge is analyzed by the finite element method using the computer program developed at the University of Colorado at Boulder (Budde et al., 1974). The bridge is shown in Fig. 11. Two types of finite element models are used: one for the bridge as it stands, and another for the bridge after the central pier has been removed.

Each bridge segment is modeled with a combination of beam and plate elements. The transverse elements utilized to simulate the behavior of beam columns and longitudinal girders are the type of eight-node finite elements which have been implemented to extend FEMAN finite element capability.

For each analysis, the foundation is fixed at one end and subjected to a static force. A general picture of the total load can be seen in these figures. Three static loadings are taken into account: (1) dead load, (2) time history input, and (3) imposed lateral loads of equal magnitude plus a constant load. The time history input represents randomly generated wave amplitudes dependent upon the properties of the soil and located below the bridge. Large wave heights are taken as 10 times the height of the bridge, and these heights are varied.

An important aspect of the analysis of bridge components is to determine the effect of each bridge component on the overall performance of the structure in dynamic loading. This result is obtained by a given procedure of analysis. The analysis is performed in three stages through a series of nonlinear analyses of individual bridge elements. The results of the first stage are presented in Fig. 12.

Fig. 12, Double Tower Cyclic Test Analysis, shows the first stage of the bridge analysis. It shows the bridge with fixed ends in all directions applied at its base. By other observations, boundary conditions were included at the base and at the center of the bridge. It is assumed that no energy is lost in any of the joints of the bridge and structural discontinuities do not exist. The calculation is based on the behavior of the bridge structure. The results of the first stage indicate the importance of proper coupling between adjacent bridge structures, as the mutual effect of the structures is great.

There are two "dead to fixed" boundary schemes, which are used for the Double Tower Cyclic Test Analysis, etc. First, the ends of the bridge are coupled in such a manner that no longitudinal movement, but no side slant of the bridge is free; the second, "fixed," the ends of the bridge longitudinally without movement, but allowing each of the towers to move independently. These two dead to fixed situations are the longitudinal boundaries, and the ends are coupled, so the two towers do not move with respect to each other.

The following results and graphs are reported for each of the different boundary conditions:

* Plots of the Structural Model

The plots of the structural model are shown in Figs. 13 through 16. The plots show the displacement of the bridge under various loading conditions. The plots are divided into two parts: (1) the displacement of the bridge under static loading conditions, and (2) the displacement of the bridge under dynamic loading conditions. The plots are divided into two parts: (1) the displacement of the bridge under static loading conditions, and (2) the displacement of the bridge under dynamic loading conditions.

EXECUTIVE SUMMARY

In preparation for the bid of the new East Spans of the San Francisco – Oakland Bay Bridge, the SC Solutions, Inc. investigated a set of preliminary designs for the Main Span Replacement Structure. The results of the time history analyses conducted for each preliminary design alternative is contained in this document. Four preliminary designs are under investigation: (i) Single Tower Cable Stay Alternative 1; (ii) Single Diamond Tower Cable Stay Alternative 2; (iii) Two Tower Cable Stay Bridge Alternative; (iv) Segmental Construction Alternative.

Each bridge alternative was modeled with a combination of truss and beam elements. The truss/cable elements were used to model the cable stays; beam elements were implemented for the tower and superstructure geometry. At the base of each tower, a [12x12] stiffness matrix was implemented to account for soil-foundation interaction.

For each analysis, the dead load is first applied statically to the structure. After application of the dead load, multi-support time history nonlinear analysis is performed. Multiple displacement time history inputs were applied at the base of each soil spring in the three orthogonal directions. Different displacement time history input motions were implemented depending on the properties of the soil located below each tower. Large displacement effects are included in both the static gravity load and time history analyses.

An important effect in the analysis of cable structures is to capture the initial stress state of the cable stays, resulting from gravity loads. The deflected geometry of the structure due to dead load must result in a level roadway surface. The effects of initial stress are properly included in these analyses through use of nonlinear material properties for the truss (cable) elements. The cables exhibit the proper level of stress after dead load is applied.

For the ‘Double Tower Cable Stay Alternative’, on the East Side of the main span, the boundary frame was modeled and motions applied at its base. For other alternatives, boundary frames were included at the East end of the main span structure. This was necessary in order to study the effects of the adjacent viaduct structures and their connections to the main span on the behavior of the bridge structure. The results of the analysis underscored the importance of proper coupling between adjacent structures, as well as the critical nature of the connection design.

Three different ‘deck to tower’ connection schemes were studied for the ‘Double Tower Cable Stay Alternative’. First, the deck is allowed to move independently from the towers in the longitudinal direction, but remains fixed to the towers in the transverse direction. Next, the deck is isolated longitudinally with soft bearings, but remains fixed to the towers in the transverse direction. Finally the deck is free to move in the longitudinal direction, and the deck is isolated in the transverse direction with soft bearings.

The following results and graphs are reported for each of the alternative structures and boundary conditions:

- Plots of the Structural Model

- Table of Extreme Tower and Foundation Forces
- Table of Tower Maximum Differential Displacements (Drifts)
- Table of Maximum Deck Displacements
- Time History Plots of Tower Differential Displacements (Drifts)

For the 'Double Tower Cable Stay Alternative' an additional study of the Diamond Tower was performed. A detailed model of the diamond tower of the 'Double Tower Cable Stay Alternative' was created using 20-noded solid elements. The first 10 modes of vibrations of this tower were extracted. A vertical load, representing the vertical component of cable forces, is applied at the top of the tower along with the weight of the tower. Next, the frequencies and modes of vibration of the tower were computed. Large displacements were used to include the geometric stiffness of the tower in the frequency analysis.

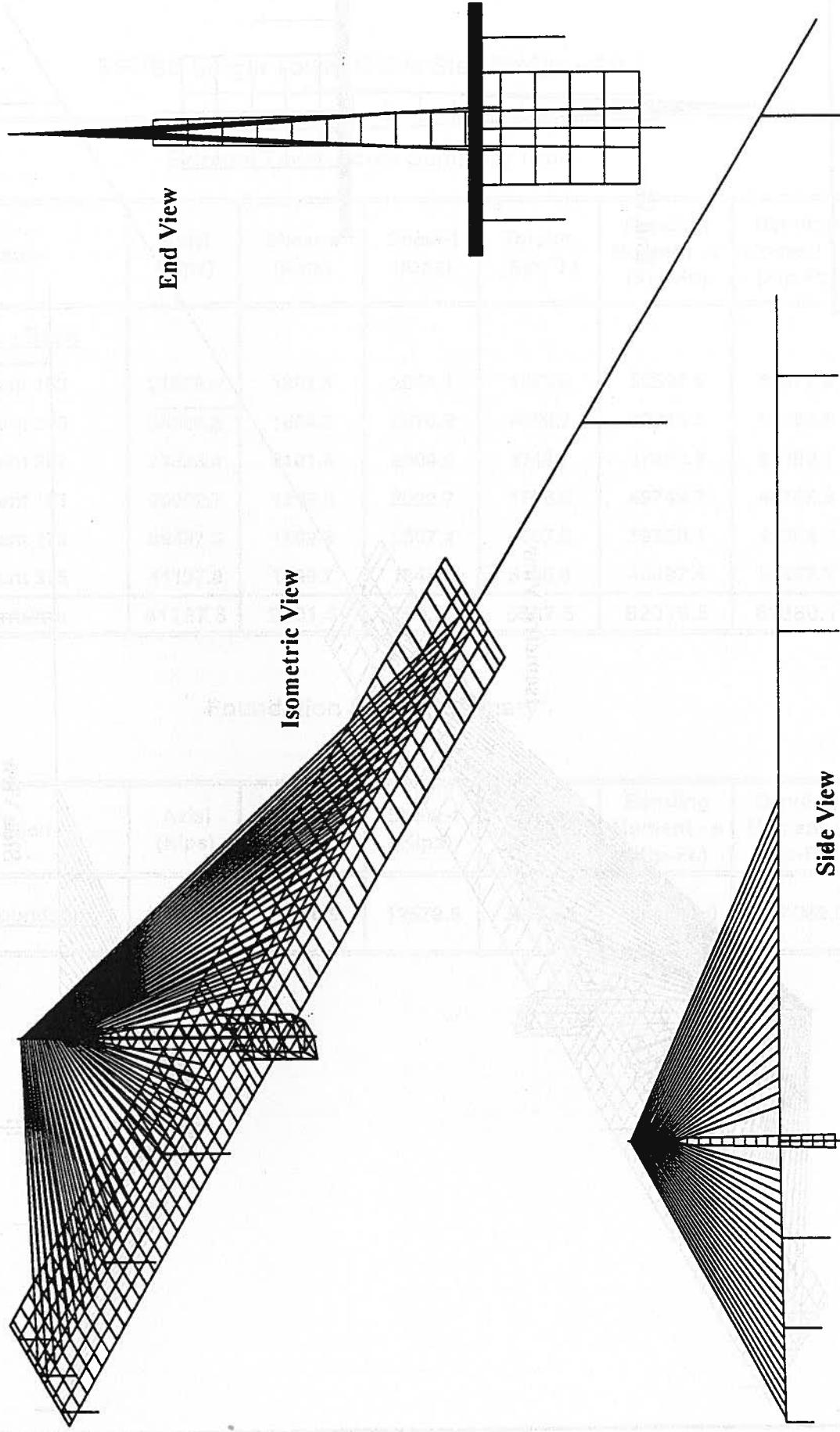
Mode shapes of this tower are plotted and the period of each mode along with its effective modal mass participation ratio is tabulated. The first mode with 53% and the second mode with 65% effective modal mass participation ratios are the two significant modes in the longitudinal and transverse directions, respectively. For each mode the deformed shape of the tower is plotted.

In order to get a general overview of the stress distribution in the tower, modal stress contours are plotted at elevations 15 ft, 65 ft, and 220 ft for the first two modes. These plots show potential locations of high stresses or high stress concentrations in the event of an earthquake. As one would expect, the first and second modes of vibrations will cause high stresses at the base of the tower and at the strut.

SFOBB Single Tower Alternative 1

Single Tower Alternative 1
Finite Element ADINA Model

PBQD/HNTB
Caltrans - DOS

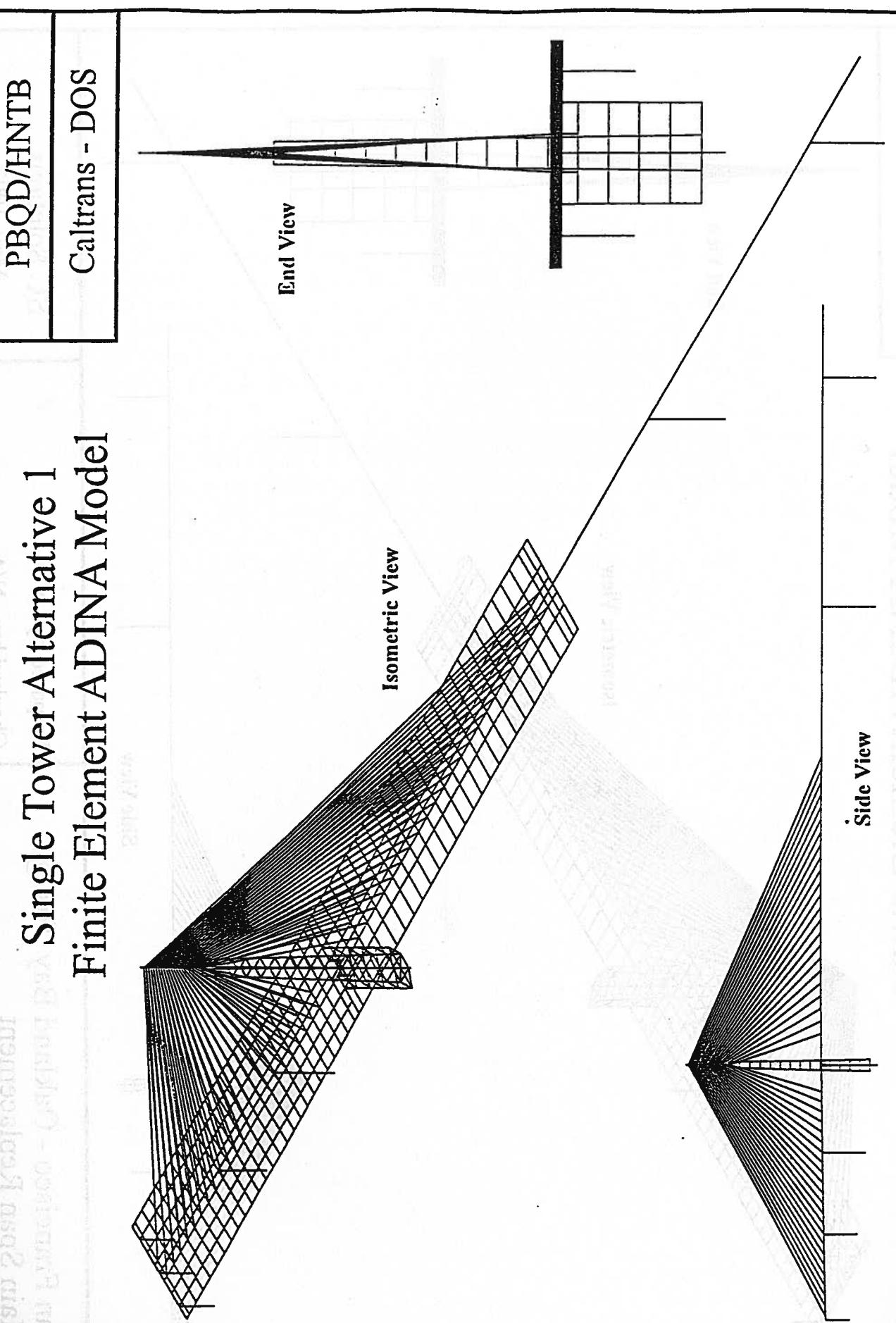


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Single Tower Alternative 1 Finite Element ADINA Model



PBQD/HNTB
Caltrans - DOS

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May 1997

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San Francisco - Oakland Bay Bridge
Main Span Replacement

SFOBB Single Tower Cable Stay Bridge - Alt 1

Extreme Tower Force Summary Table

Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
<u>Tower - Base</u>						
Element 163	21656.4	1397.1	2041.1	1693.4	50597.5	52971.6
Element 263	35504.5	1664.2	2570.2	4459.7	62019.5	47286.6
Element 207	34533.4	2101.4	2004.9	2748.7	47423.7	61380.1
Element 171	20002.7	1305.0	2022.7	1778.9	49749.7	48746.8
Element 179	39487.6	1599.8	2507.4	6567.5	59868.1	46262.2
Element 235	41137.8	1996.7	1845.6	5466.9	44997.4	59997.3
Maximums	41137.8	2101.4	2570.2	6567.5	62019.5	61380.1

Foundation Force Summary

Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
Tower Foundation	115084.5	11970.9	12579.6	38565.1	1648781.0	1416982.0

SFOBB Single Tower Cable Stay Bridge - Alt 1

Extreme Tower Force Summary Table						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
Tower - Below Deck						
Element 178	12885.3	656.5	1753.3	1092.7	26229.0	7127.7
Element 214	30948.5	2344.0	2491.5	2851.2	35672.9	37543.0
Element 270	33492.6	1523.3	3616.4	3283.7	56069.6	20407.2
Element 242	33675.1	2279.3	2203.0	4939.5	33419.3	36699.9
Element 170	12304.0	691.5	2058.0	955.3	31378.5	7751.2
Element 186	34611.5	1546.6	3281.3	5614.6	50567.5	20452.4
Maximums	34611.5	2344.0	3616.4	5614.6	56069.6	37543.0

SFOBB Single Tower Cable Stay Bridge - Alt 1

Extreme Tower Force Summary Table						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
Tower - Above Deck						
Element 187	34867.0	1022.8	1205.7	2903.7	28045.4	17411.0
Element 215	32857.4	886.5	1381.5	3972.0	31479.7	14931.1
Element 271	35532.5	1049.2	1188.6	3783.6	29187.7	17509.8
Element 243	36479.4	866.4	1445.1	2612.9	32450.9	14237.4
Maximums	36479.4	1049.2	1445.1	3972.0	32450.9	17509.8

SFOBB Single Tower Cable Stay Bridge - Alt. 1

Absolute Maximum Tower Differential Displacement Summary Table

Location*	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
at Deck Elevation	5.8	0.7	4.7
at Top of Tower	13.6	4.7	31.1

* Note: All differential displacements are calculated between the tower base and the specified locations.

**SFOBB Single Tower
Cable Stay Bridge - Alt. 1**

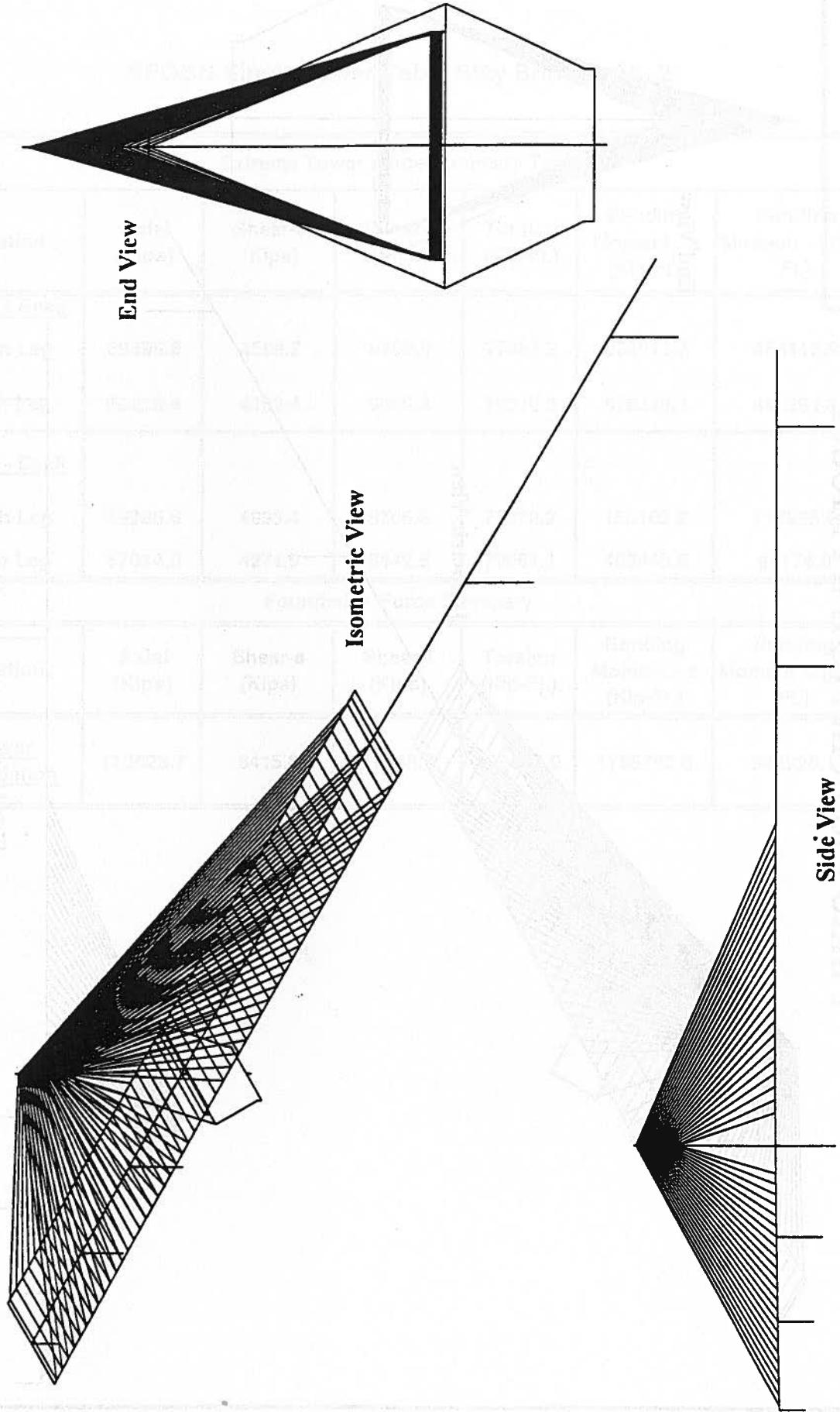
Displacement Summary Table			
Location	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>West End</u>			
South Edge	18.6	7.0	10.2
Centerline	21.0	8.3	13.9
North Edge	20.3	7.0	10.2
<u>East End</u>			
South Edge (Bridge Node)	31.7	22.7	42.2
Centerline (Bridge Node)	25.6	23.2	42.2
North Edge (Bridge Node)	20.3	30.9	42.3

SFOBB Single Tower Alternative 2

Single Tower Alternative 2
Finite Element ADINA Model

PBQD/HNTB

Caltrans - DOS



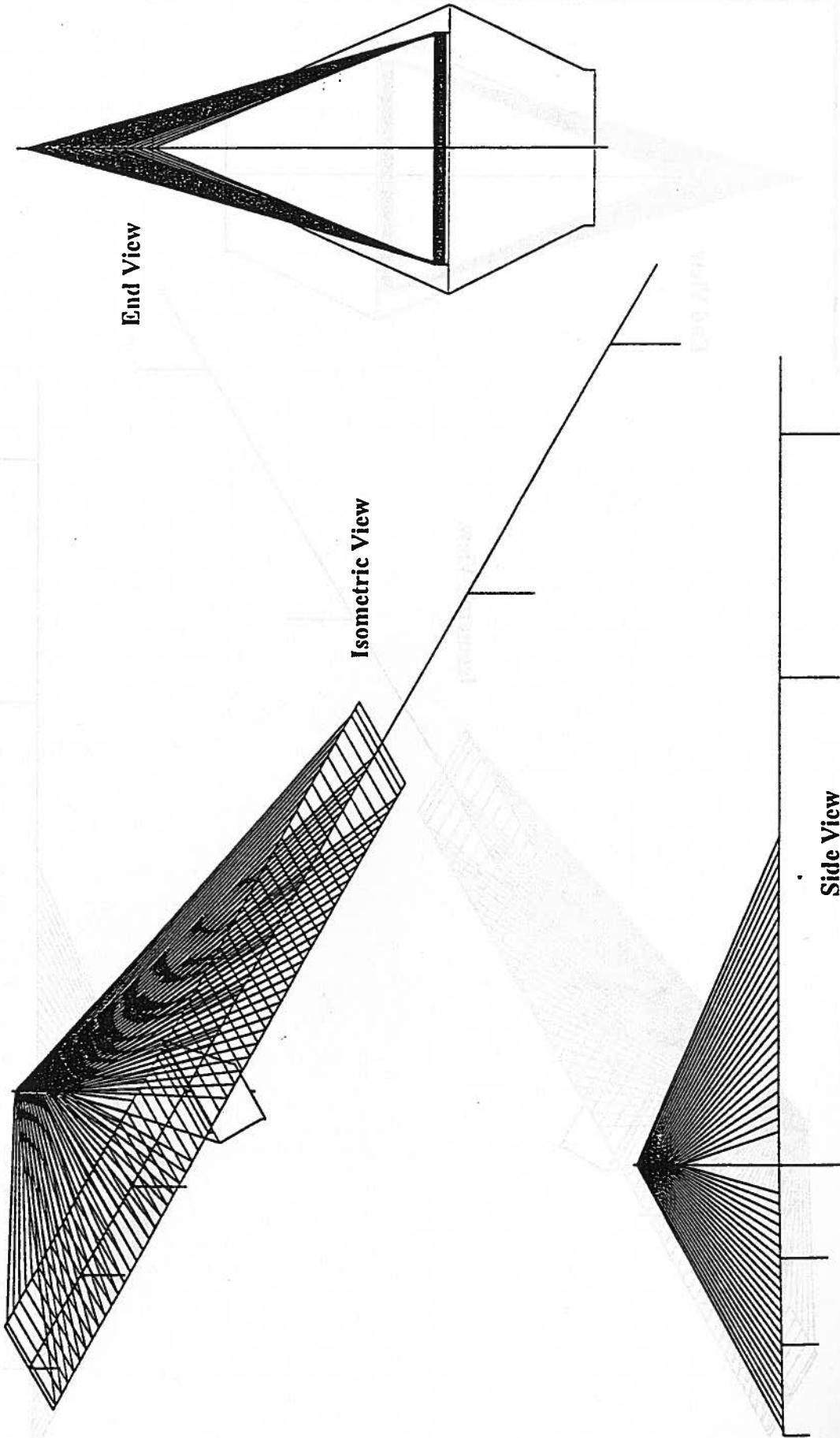
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Single Tower Alternative 2
Finite Element ADINA Model

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Caltrans - DOS



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Main Span Replacement

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SFOBB Single Tower Cable Stay Bridge - Alt. 2

Extreme Tower Force Summary Table						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip Ft.)
<u>Tower - Base</u>						
South Leg	59395.6	4589.7	8469.5	77493.2	633913.7	459118.9
North Leg	60278.4	4162.4	9809.4	79318.8	676349.1	440961.1
<u>Tower - Deck</u>						
South Leg	55960.6	4693.4	8706.8	77272.2	150103.2	115955.9
North Leg	57014.0	4271.9	8442.5	79061.1	409440.8	98174.9
Foundation Force Summary						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip Ft.)
<u>Tower Foundation</u>	112828.7	8415.8	12545.7	256007.0	1755793.0	999925.1

SFOBB Single Diamond Tower Cable Stay Bridge - Alt. 2

Absolute Maximum Tower Differential Displacement Summary Table

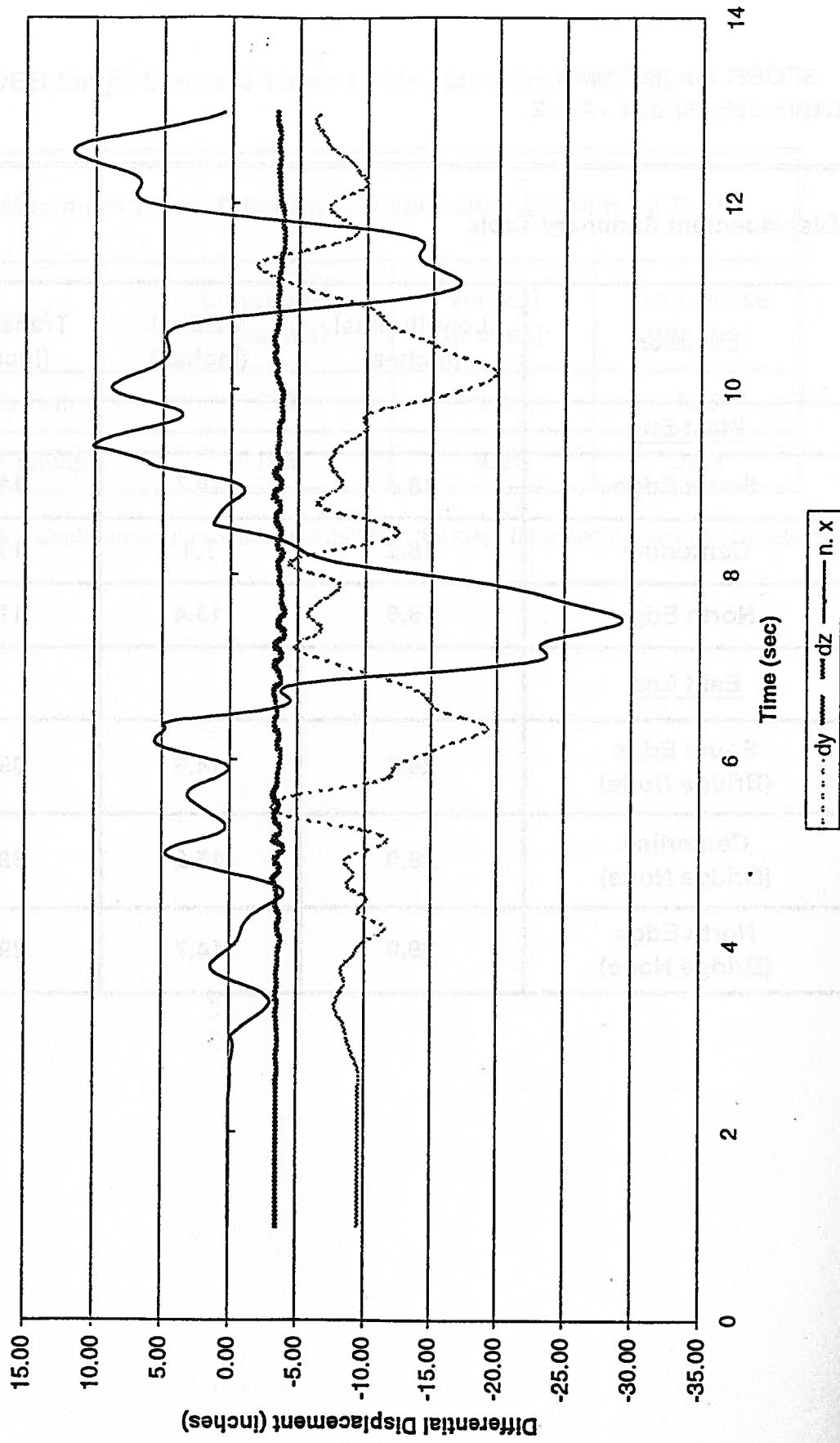
Location*	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
at Deck Elevation	7.3	1.3	12.7
at Top of Tower	19.9	4.0	29.3

*Note: All differential displacements are calculated between the tower base and the specified locations.

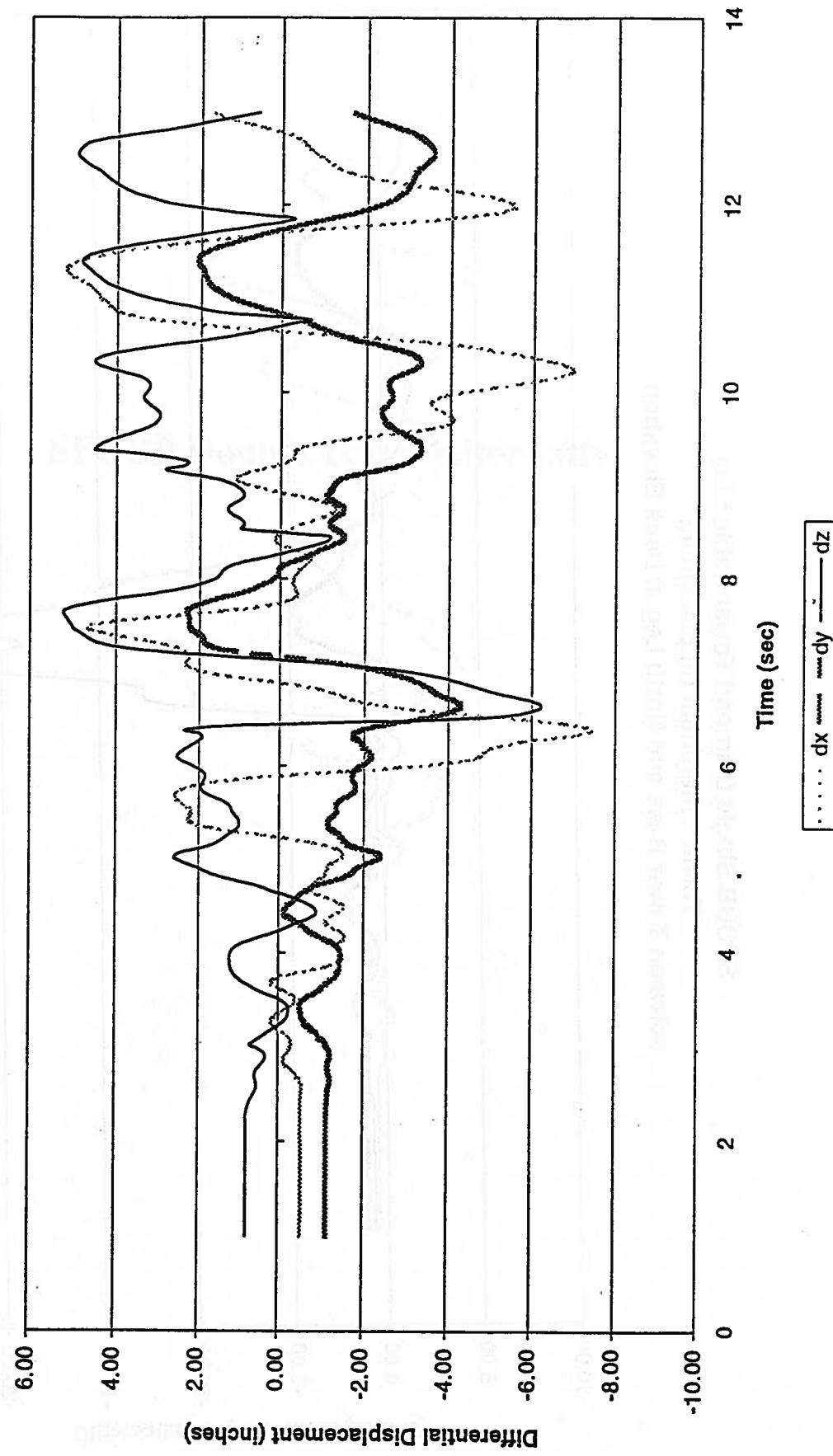
**SFOBB Single Tower
Cable Stay Bridge - Alt. 2**

Displacement Summary Table			
Location	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>West End</u>			
South Edge	18.3	15.7	11.5
Centerline	18.2	7.1	11.5
North Edge	16.6	13.4	11.6
<u>East End</u>			
South Edge (Bridge Node)	28.1	14.6	39.6
Centerline (Bridge Node)	29.0	15.2	39.6
North Edge (Bridge Node)	29.0	14.7	39.6

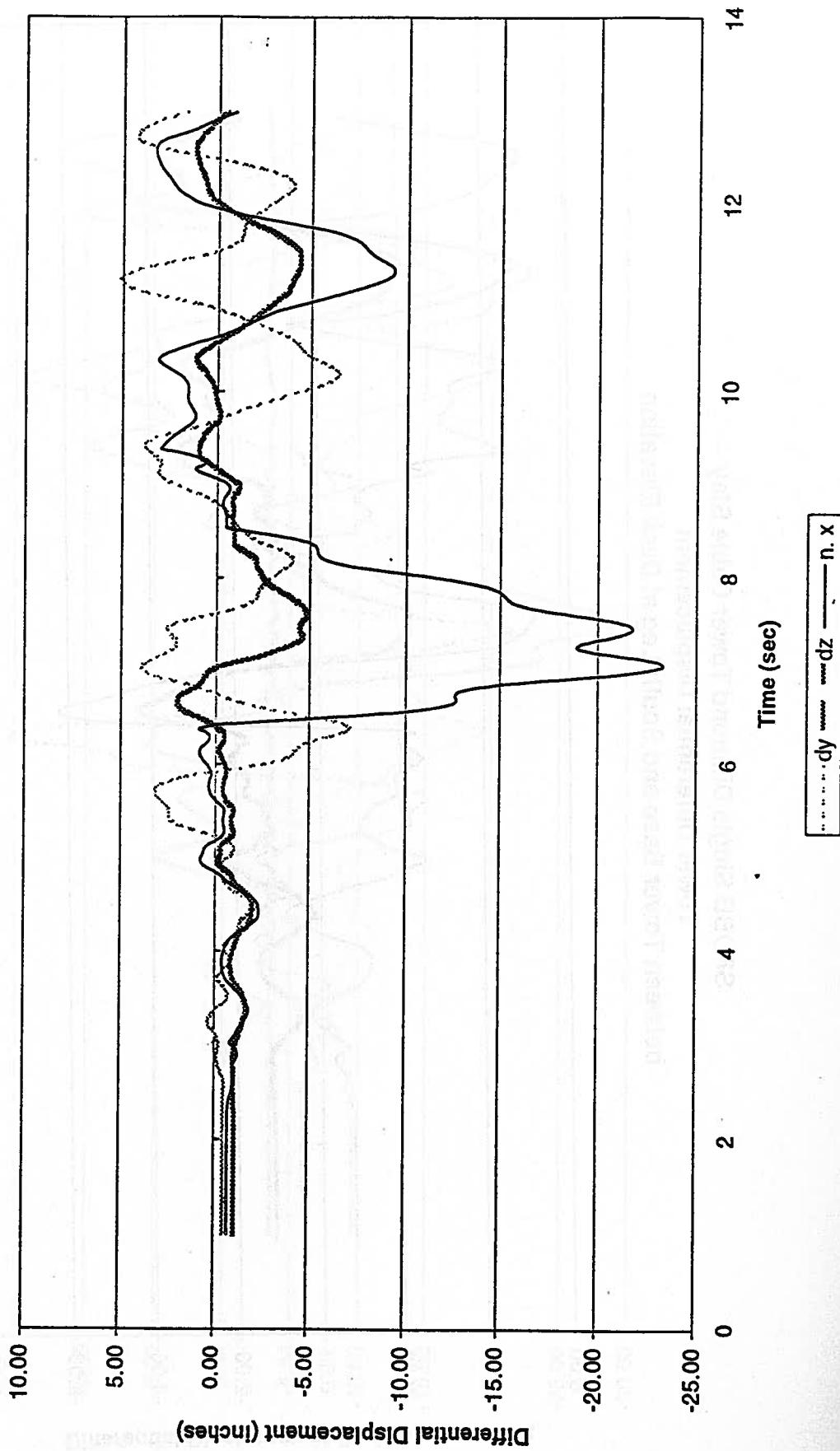
**SFOBB Single Diamond Tower Cable Stay
Tower Differential Displacement
between Tower Base and Top of Tower**



SFOBB Single Diamond Tower Cable Stay
Tower Differential Displacement
between Tower Base and South Leg at Deck Elevation



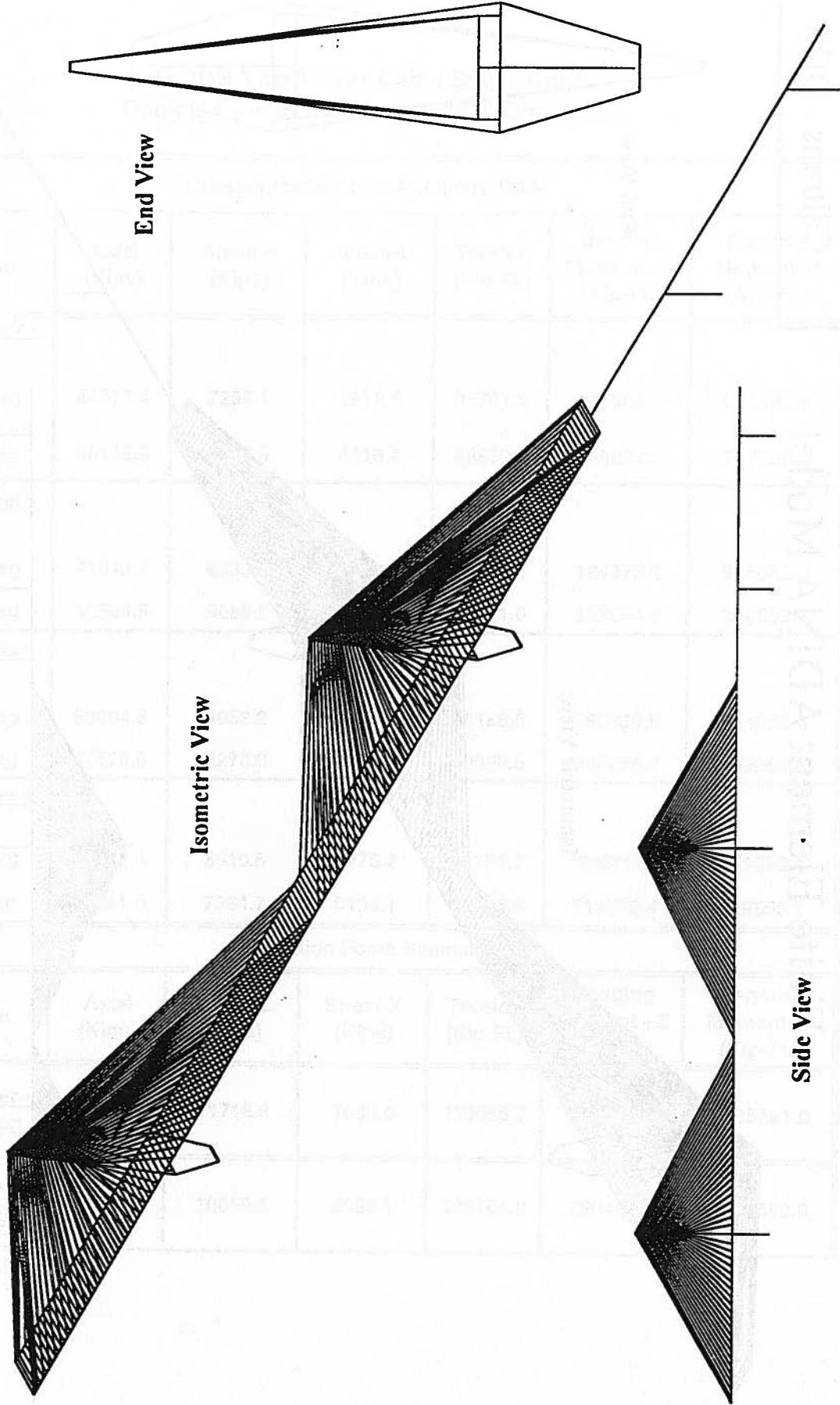
SFOBB Single Diamond Tower Cable Stay
Tower Differential Displacement
between Tower Base and North Leg at Deck Elevation



SFOBB Double Tower Alternative

Two Tower Alternative Finite Element ADINA Model

PBQD/HNTB
Caltrans - DOS



San Francisco - Oakland Bay Bridge
Main Span Replacement

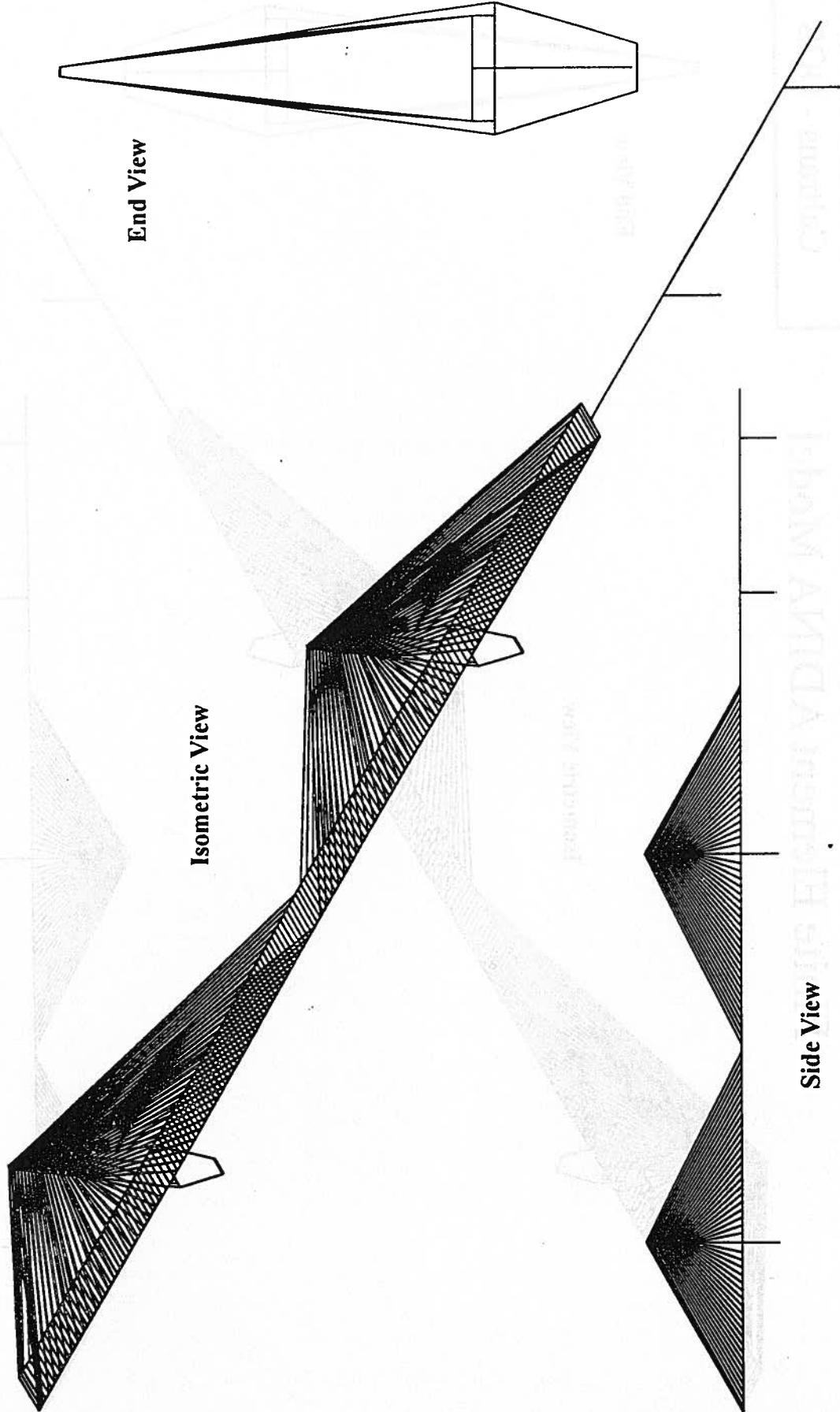
Prepared by: N/A
Checked by: N/A

SC Solutions
May 1997

Two Tower Alternative
Finite Element ADINA Model

PBQD/HNTB

Caltrans - DOS



San Francisco - Oakland Bay Bridge
Main Span Replacement

Prepared by: N/A
Checked by: N/A

SC Solutions
May 1997

SFOBB Two Tower Cable Stay Bridge
Deck is Free in the Longitudinal Direction

Extreme Tower Force Summary Table						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
<u>West Tower - Base</u>						
South Leg	44977.4	7296.1	6912.6	33761.2	817965.7	669192.8
North Leg	50188.5	8636.5	4016.4	48880.5	749237.3	767225.2
<u>West Tower - Deck</u>						
South Leg	41313.2	6535.4	6732.3	32683.1	124673.1	215662.8
North Leg	46599.5	8088.5	3690.1	49751.0	122664.8	250893.1
<u>East Tower - Base</u>						
South Leg	53004.8	9053.2	3626.9	46188.8	580830.6	841056.0
North Leg	57528.0	8273.0	4758.2	50098.5	600065.3	732262.6
<u>East Tower - Deck</u>						
South Leg	49703.4	8610.5	3775.7	46171.7	94078.7	251850.1
North Leg	54051.0	7384.7	5136.1	50095.2	111372.4	229663.7
Foundation Force Summary						
Location	Axial (Kips)	Shear-Z (Kips)	Shear-X (Kips)	Torsion (Kip-Ft.)	Bending Moment - Z (Kip-Ft.)	Bending Moment - X (Kip-Ft.)
<u>West Tower - Foundation</u>	80752.1	11716.8	7033.0	126085.7	640076.6	1908791.0
<u>East Tower - Foundation</u>	96751.4	10059.8	8083.1	235104.0	720483.6	2332689.0

SFOBB Two Tower Cable Stay Bridge
Deck is Free in Longitudinal Direction

Absolute Maximum Tower Differential Displacement Summary Table

Location*	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>West Tower</u>			
at Deck Elevation	2.8	0.6	8.4
at Top of Tower	18.5	2.0	30.9
<u>East Tower</u>			
at Deck Elevation	3.0	0.7	4.6
at Top of Tower	21.1	2.5	38.3

*Note: All differential displacements are calculated between the tower base and the specified locations.

SFOBB Two Tower Cable Stay Bridge
Deck is Isolated in Longitudinal Direction and Fixed Transversely

Extreme Tower Force Summary Table						
Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip Ft.)
<u>West Tower - Base</u>						
South Leg	44977.4	7296.1	6912.6	33761.2	817965.7	669192.8
North Leg	50188.5	8636.5	4016.4	48880.5	749237.3	767225.2
<u>West Tower - Deck</u>						
South Leg	41313.2	6535.4	6732.3	32683.1	124673.1	215662.8
North Leg	46599.5	8088.5	3690.1	49751.0	122664.8	250893.1
<u>East Tower - Base</u>						
South Leg	53004.8	9053.2	3626.9	46188.8	580830.6	841056.0
North Leg	57528.0	8273.0	4758.2	50098.5	600065.3	732262.6
<u>East Tower - Deck</u>						
South Leg	49703.4	8610.5	3775.7	46171.7	94078.7	251850.1
North Leg	54051.0	7384.7	5136.1	50095.2	111372.4	229663.7
Foundation Force Summary						
Location	Axial (Kips)	Shear-Z (Kips)	Shear-X (Kips)	Torsion (Kip-Ft.)	Bending Moment - Z (Kip-Ft.)	Bending Moment - X (Kip Ft.)
<u>West Tower - Foundation</u>	81194.9	11607.9	13055.6	98634.8	1522942.0	1898624.0
<u>East Tower - Foundation</u>	96706.3	9999.0	9092.2	217237.3	1147799.0	2359657.0

SFOBB Two Tower Cable Stay Bridge
Isolated in Longit. Direction; Fixed in Trans. Direction

Absolute Maximum Tower Differential Displacement Summary Table

Location*	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>West Tower</u>			
at Deck Elevation	6.9	0.6	8.3
at Top of Tower	23.9	2.0	30.8
<u>East Tower</u>			
at Deck Elevation	5.0	0.7	8.5
at Top of Tower	10.6	2.5	38.6

*Note: All differential displacements are calculated between the tower base and the specified locations.

SFOBB Two Tower Cable Stay Bridge
Deck is Isolated in Longitudinal Direction and Fixed Transversely

Deck Displacement Summary Table			
Location	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>West End</u>			
South Edge	22.0	3.3	29.3
North Edge	22.9	2.7	29.3
<u>East End</u>			
South Edge (Bridge Node)	22.6	58.7	36.0
North Edge (Bridge Node)	22.9	58.7	36.0
<u>East End</u>			
Boundary Frame Node	23.1	58.7	36.0

SFOBB Segmental Alternative

**Segmental Alternative
Finite Element ADINA Model**

PBQD/HNTB

Caltrans - DOS

Isometric View

Side View

**San Francisco - Oakland Bay Bridge
Main Span Replacement**

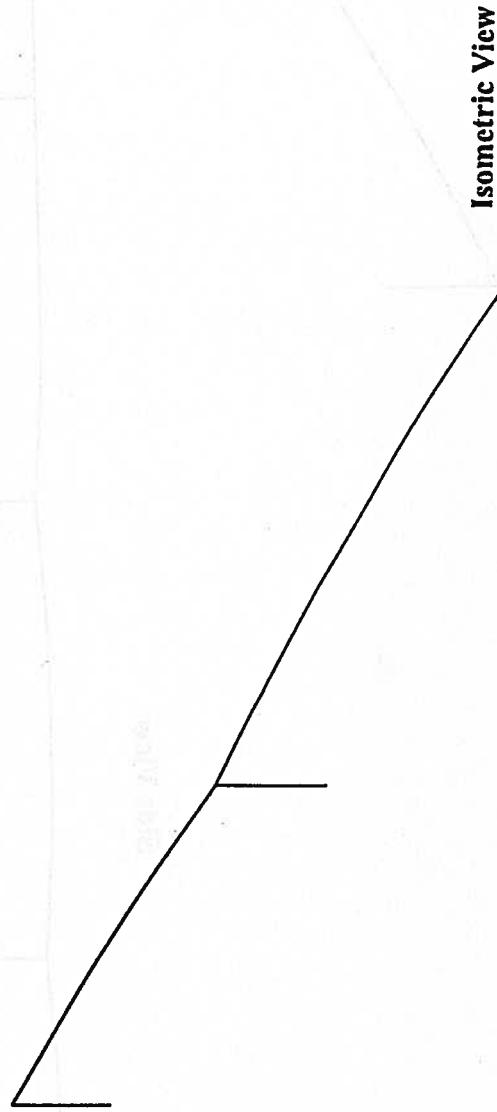
Prepared by: N/A
Checked by: N/A

SC Solutions
May 1997

**Segmental Alternative
Finite Element ADINA Model**

PBQD/HNTB

Caltrans - DOS



Isometric View

Side View



San Francisco - Oakland Bay Bridge
Main Span Replacement

Prepared by: N/A
Checked by: N/A

SC Solutions

May 1997

SFOBB Segmental Alternative

Extreme Tower Force Summary Table

Location	Axial (Kips)	Shear-s (Kips)	Shear-t (Kips)	Torsion (Kip-Ft.)	Bending Moment - s (Kip-Ft.)	Bending Moment - t (Kip-Ft.)
<u>Tower - Bases</u>						
West Tower	46823.7	10936.6	7752.1	74042.9	1324998.0	995657.5
East Tower	53214.2	9032.6	10480.2	52385.6	1876326.0	859138.5
Maximums	53214.2	10936.6	10480.2	74042.9	1876326.0	995657.5
<u>Tower - Tops</u>						
West Tower	43050.1	10875.0	7339.8	73769.3	610453.7	969835.4
East Tower	46883.3	9003.8	10194.4	51977.1	893334.0	649549.0
Maximums	46883.3	10875.0	10194.4	73769.3	893334.0	969835.4

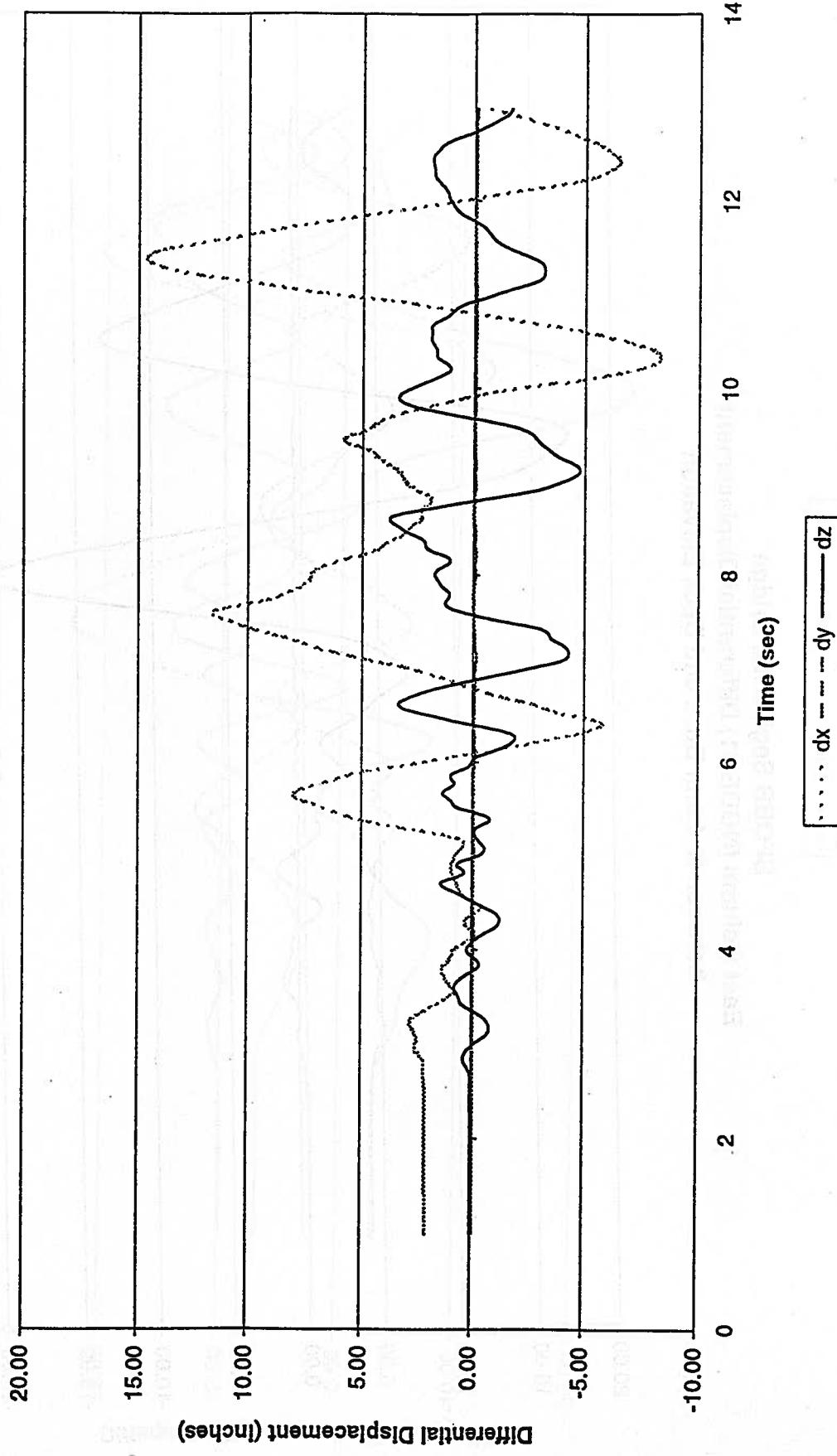
SFOBB Segmental Bridge

Absolute Maximum Tower Differential Displacement Summary Table

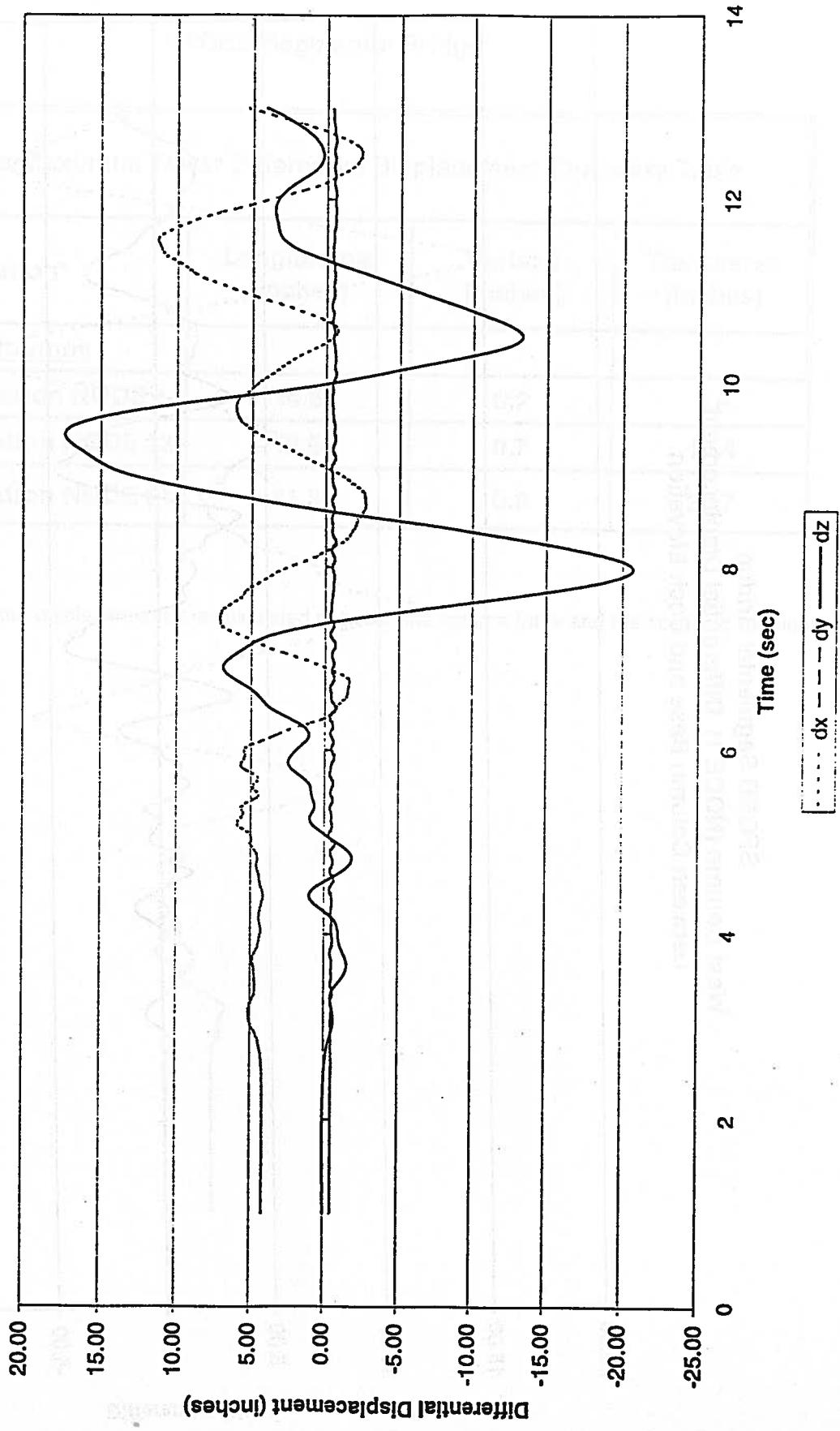
Location*	Longitudinal (Inches)	Vertical (Inches)	Transverse (Inches)
<u>East Coumns</u>			
at Deck Elevation NODE 1	14.6	0.2	4.8
at Deck Elevation NODE 23	13.5	0.7	14.4
at Deck Elevation NODE 61	11.3	0.8	20.7

*Note: All differential displacements are calculated between the column base and the specified locations.

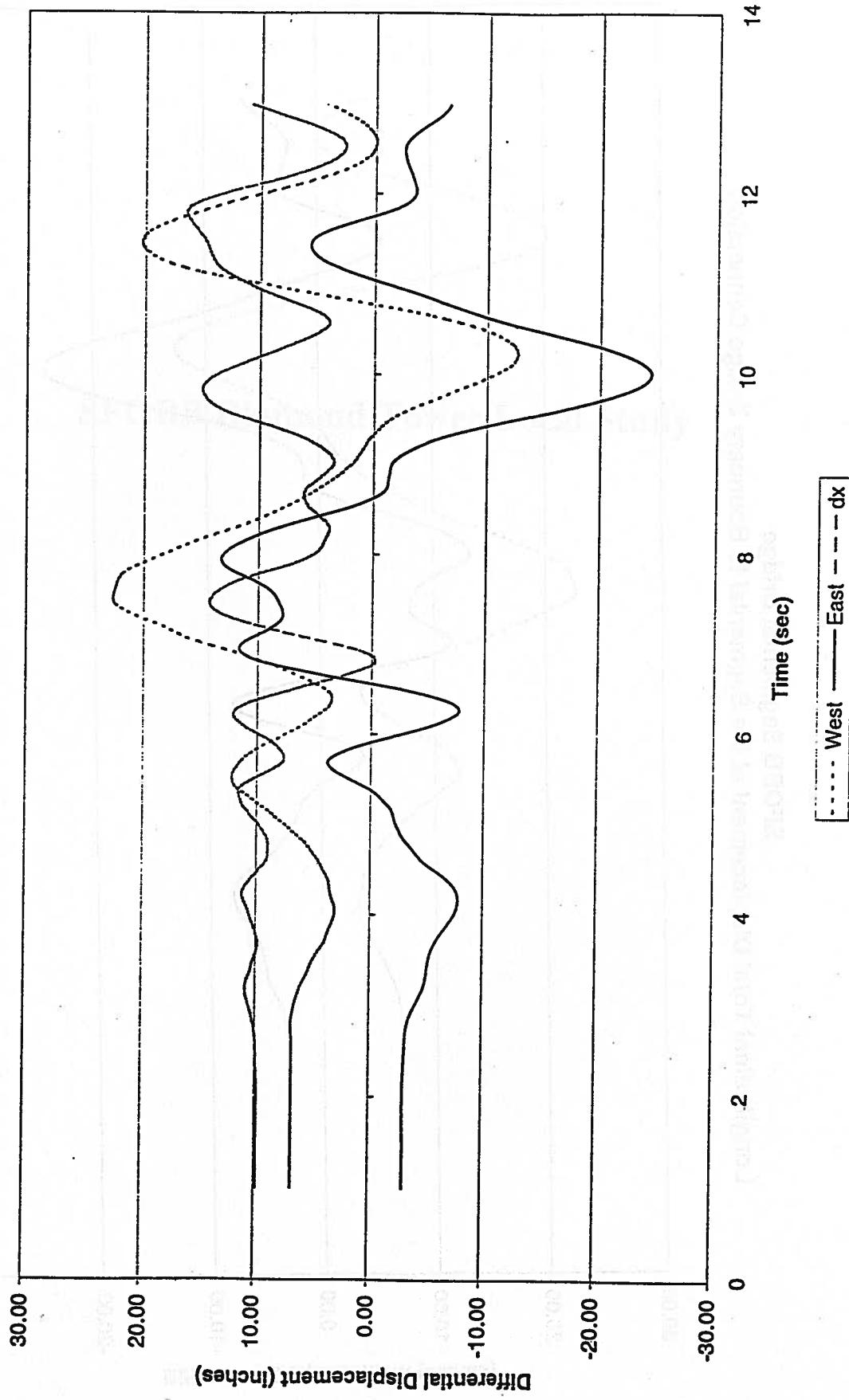
SFOBB Segmental Bridge
West Column (NODE 1) Differential Displacement
between Column Base and Deck Elevation



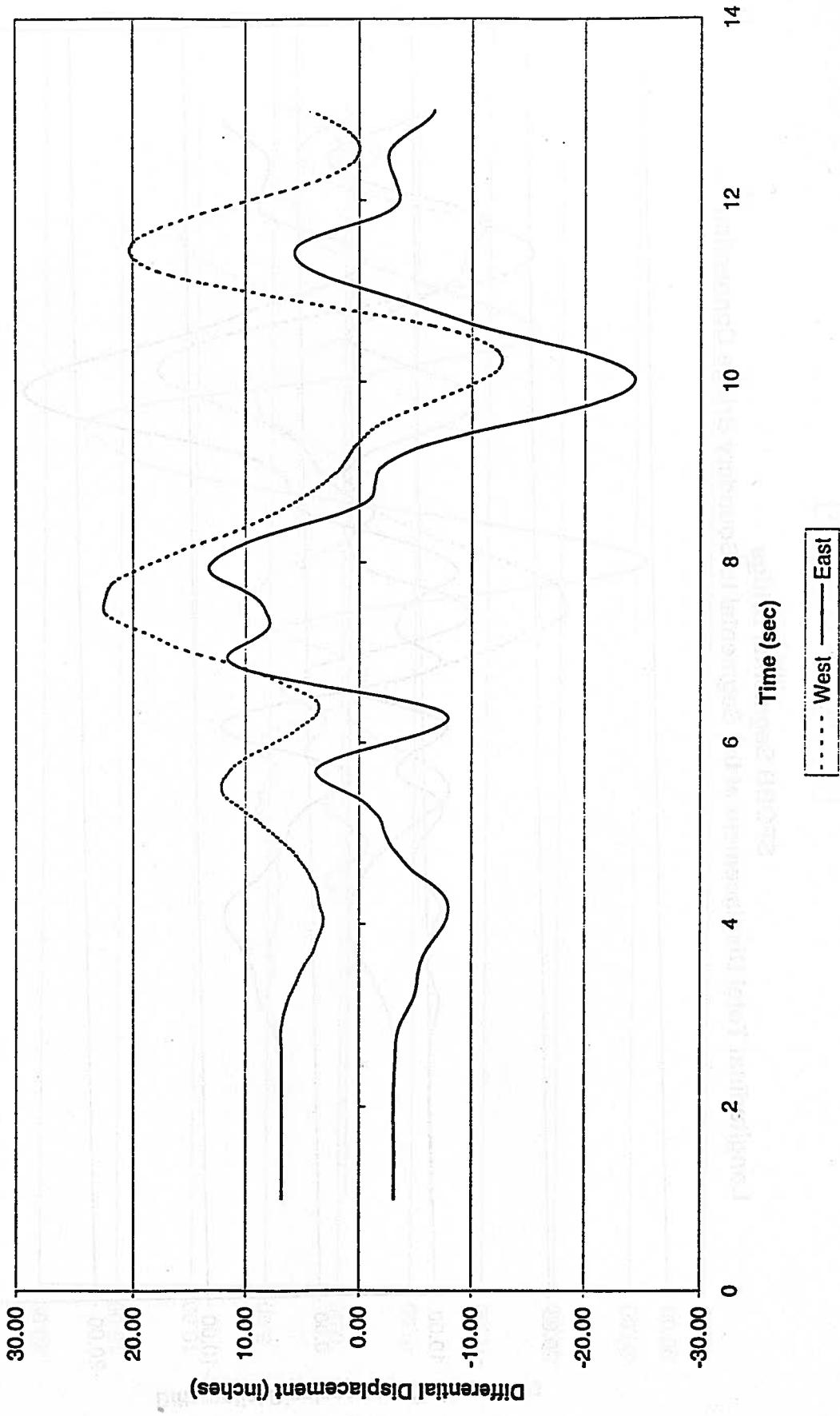
SFOBB Segmental Bridge
East Column (NODE 61) Differential Displacement
between Column Base and Deck Elevation



**SFOBB Segmental Bridge
Longitudinal Total Displacement at the Segmental to Boundary Bridge Connection**



SFOBB Segmental Bridge
Longitudinal Total Displacement at the Segmental to Boundary Bridge Connection



SFOBB Diamond Tower Local Study

SFOBB - Periods of Vibrations and Mass Participating Factor of the Diampnd Tower

Mode	Period (sec)	Modal Mass Participating Factor			Accumulative Modal Mass Participating Factor		
		X	Y	Z	X	Y	Z
1	6.74	0.53	0.00	0.00	0.53	0.00	0.00
2	3.03	0.00	0.65	0.00	0.53	0.65	0.00
3	0.94	0.00	0.00	0.01	0.53	0.65	0.01
4	0.82	0.00	0.00	0.00	0.53	0.65	0.01
5	0.79	0.20	0.00	0.00	0.73	0.65	0.01
6	0.70	0.00	0.01	0.00	0.73	0.66	0.01
7	0.51	0.00	0.08	0.00	0.73	0.74	0.01
8	0.34	0.00	0.00	0.00	0.73	0.74	0.02
9	0.33	0.00	0.00	0.00	0.73	0.74	0.02
10	0.30	0.08	0.00	0.00	0.81	0.74	0.02

X is Longitudinal

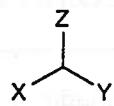
Y is Transverse

Z is Vertical

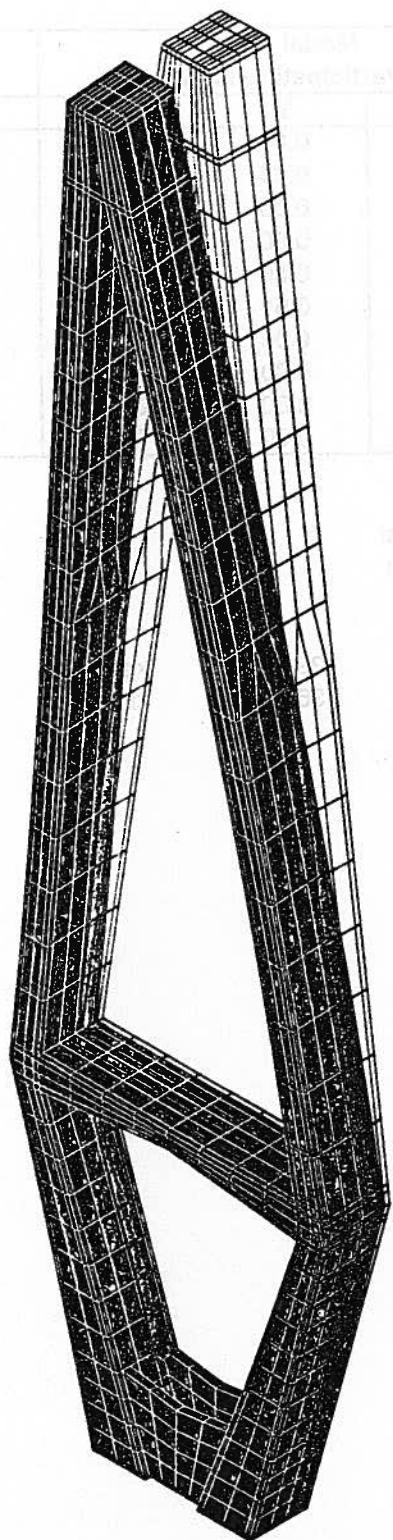
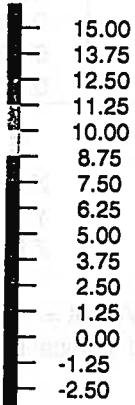
Total Weight = 22030 kips
 Applied Vertical Load= 36600 kips

ADINA

MODE 1, TIME 4.000

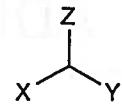


SMOOTHED
SIGMA-P1
RST CALC
MODE 1,
TIME 4.000

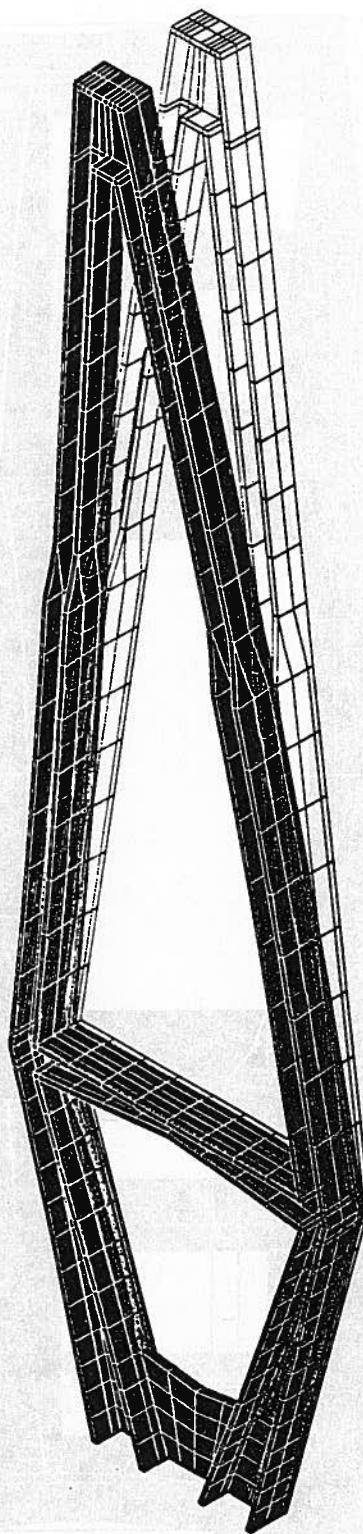
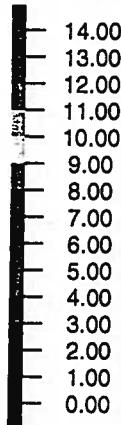


ADINA

MODE 1, TIME 4.000



SMOOTHED
SIGMA-P1
RST CALC
MODE 1,
TIME 4.000



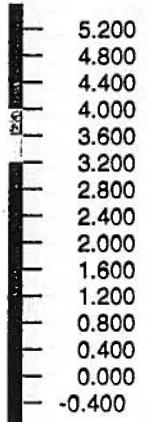
ADINA

MODE 1, TIME 4.000



Y
Z
X

SMOOTHED
SIGMA-P1
RST CALC
MODE 1,
TIME 4.000



elevation 15 ft

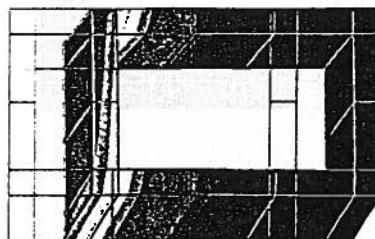
ADINA

MODE 1, TIME 4.000

Y
Z — X

SMOOTHED
SIGMA-P1
RST CALC
MODE 1,
TIME 4.000

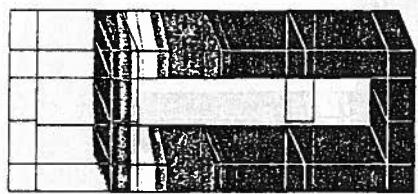
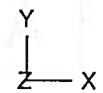
7.000
6.500
6.000
5.500
5.000
4.500
4.000
3.500
3.000
2.500
2.000
1.500
1.000
0.500
0.000



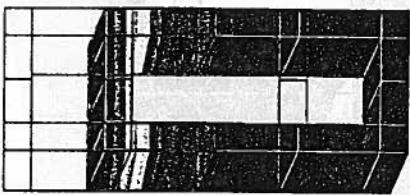
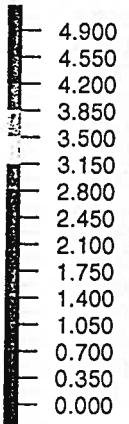
elevation 65 ft

ADINA

MODE 1, TIME 4.000



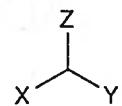
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SIGMA-P1
RST CALC
MODE 1,
TIME 4.000



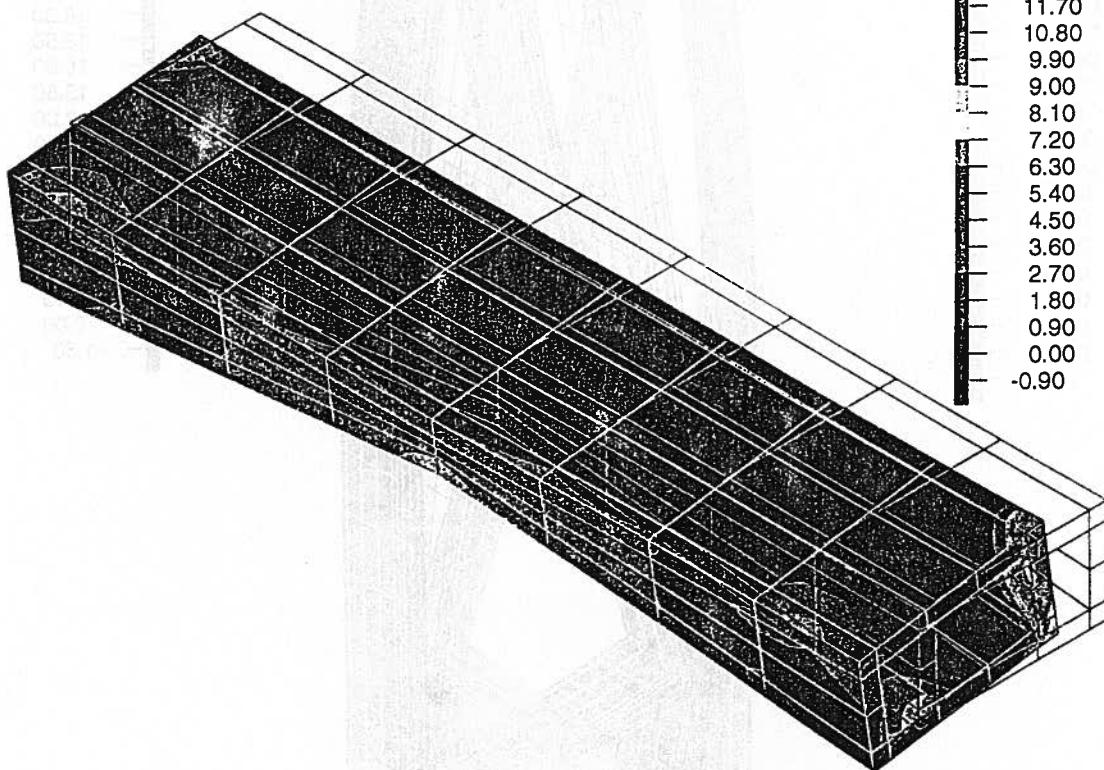
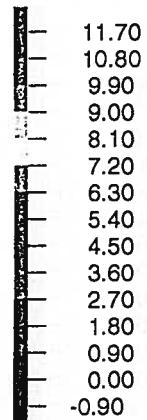
elevation 220 ft

ADINA

MODE 1, TIME 4.000



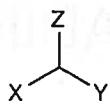
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SIGMA-P1
RST CALC
MODE 1,
TIME 4.000



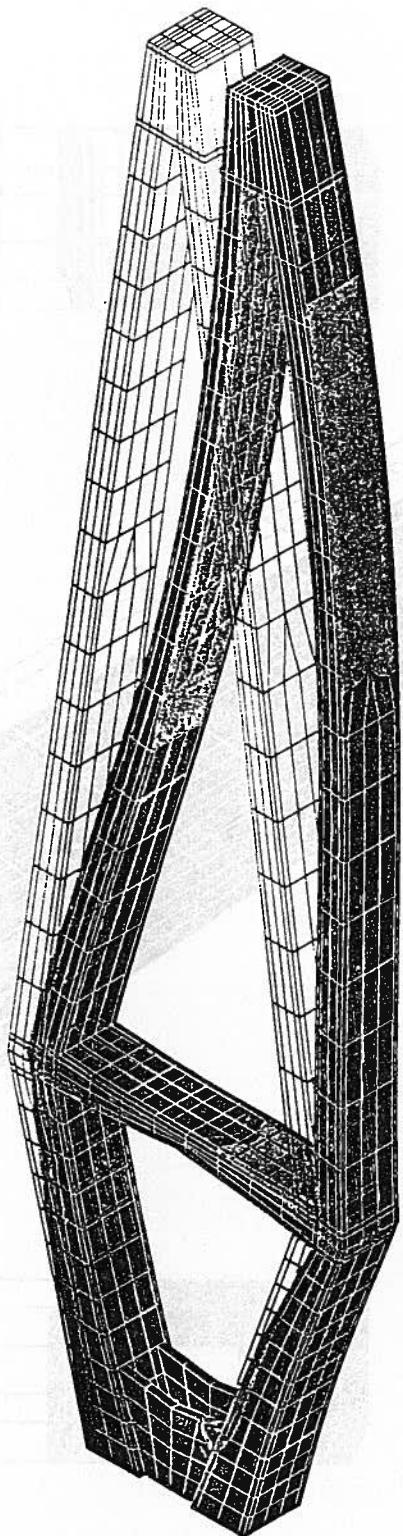
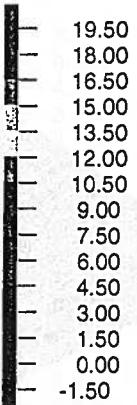
elevation 120 -132 ft

ADINA

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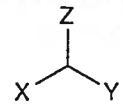


SMOOTHED
SIGMA-P1
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MODE 2,
TIME 4.000

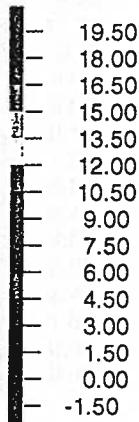


ADINA

MODE 2, TIME 4.000

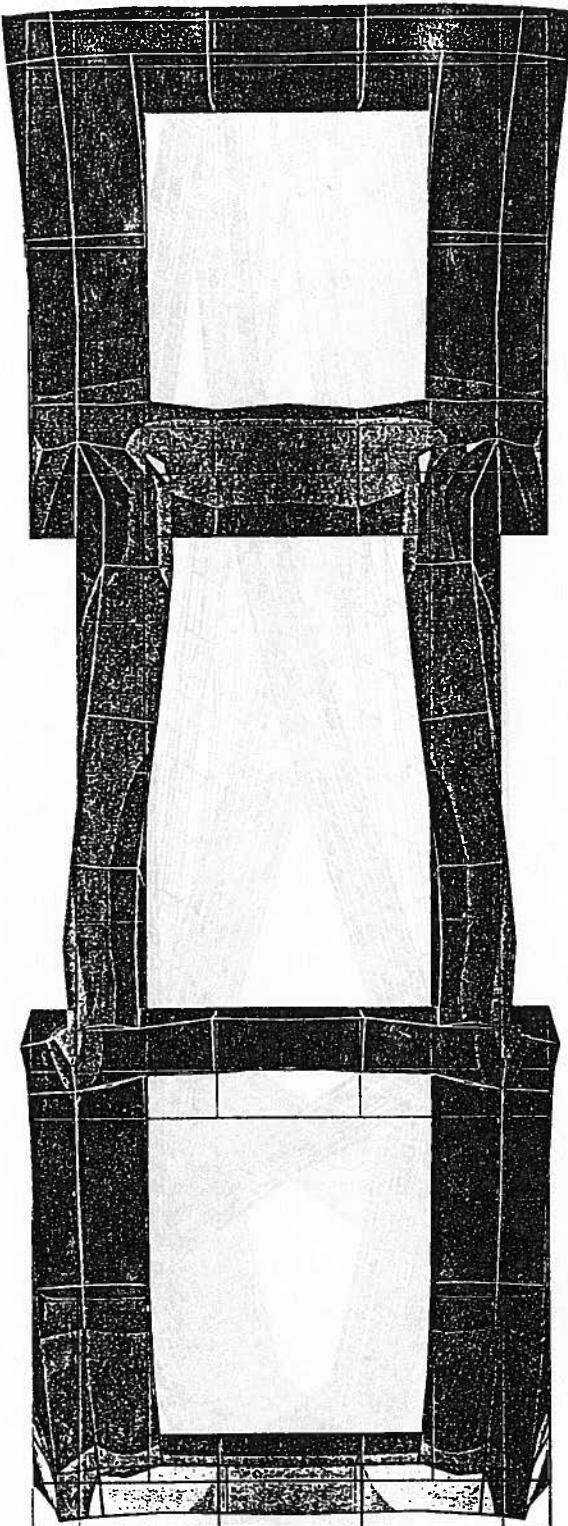


SMOOTHED
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MODE 2,
TIME 4.000

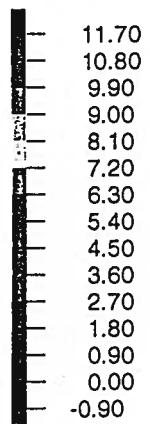


ADINA

MODE 2, TIME 4.000



SMOOTHED
SIGMA-P1
RST CALC
MODE 2,
TIME 4.000

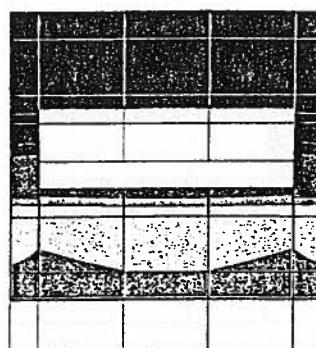


elevation 15 ft

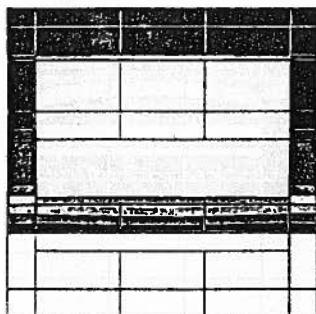
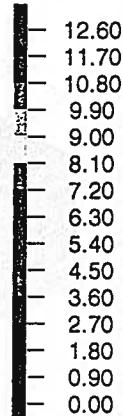
ADINA

MODE 2, TIME 4.000

Y
Z — X



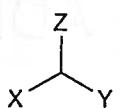
SMOOTHED
SIGMA-P1
RST CALC
MODE 2,
TIME 4.000



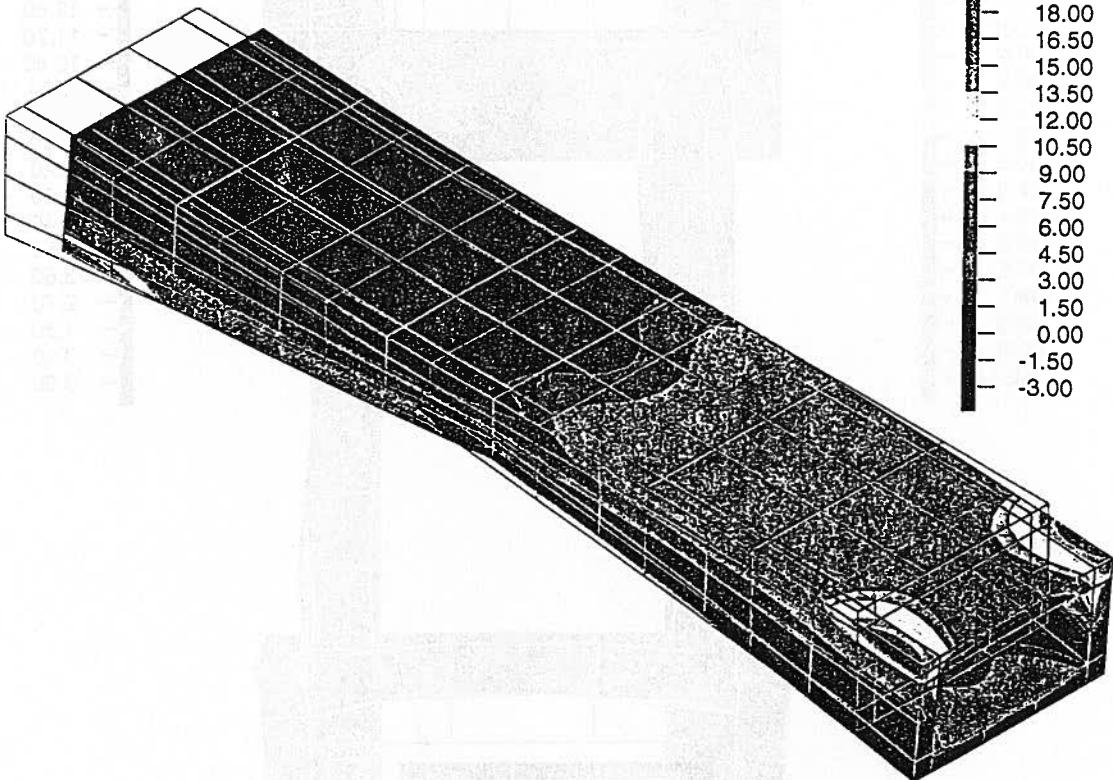
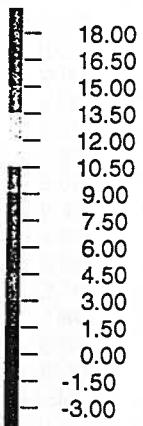
elevation 65 ft

ADINA

MODE 2, TIME 4.000



SMOOTHED
SIGMA-P1
RST CALC
MODE 2,
TIME 4.000



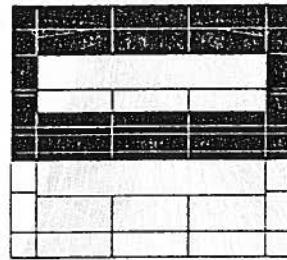
elevation 120 - 132 ft

11.30 inches/ft

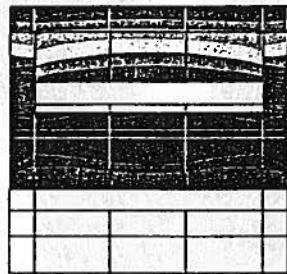
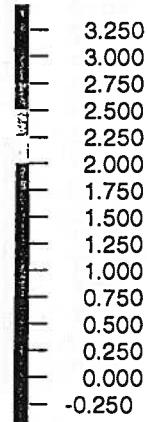
ADINA

MODE 2, TIME 4.000

Y
Z — X



SMOOTHED
SIGMA-P1
RST CALC
MODE 2,
TIME 4.000



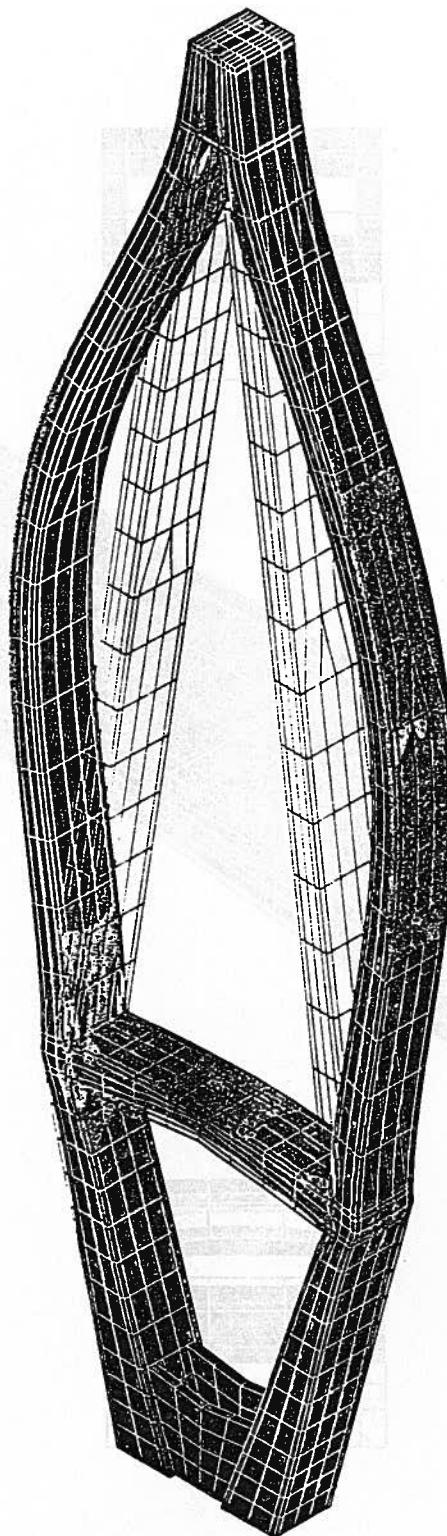
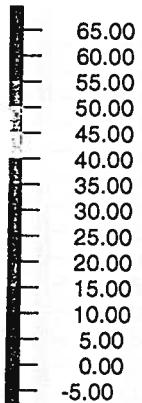
elevation 220 ft

ADINA

MODE 3, TIME 4.000

X Y Z

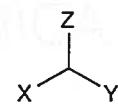
SMOOTHED
SIGMA-P1
RST CALC
MODE 3,
TIME 4.000



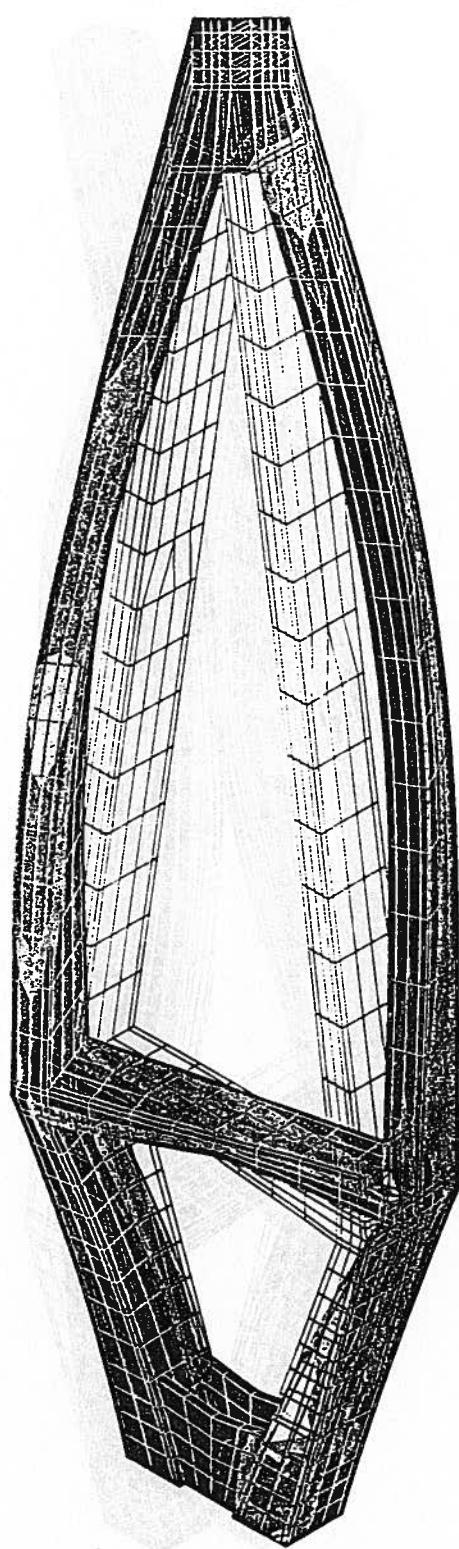
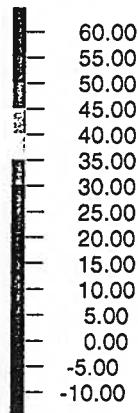
1055 convolve

ADINA

MODE 4, TIME 4.000

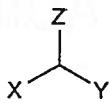


SMOOTHED
SIGMA-P1
RST CALC
MODE 4,
TIME 4.000

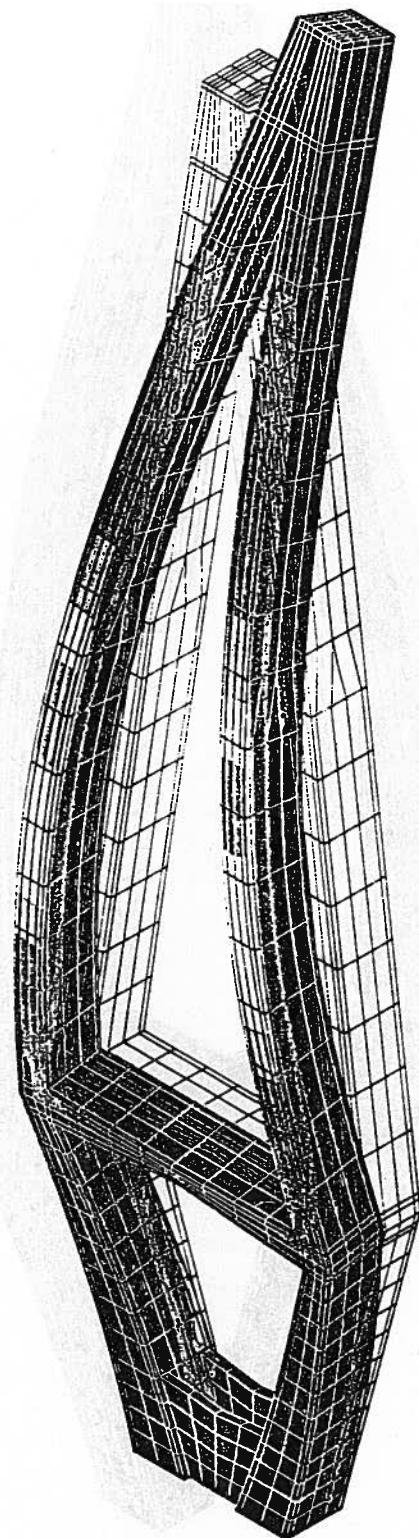
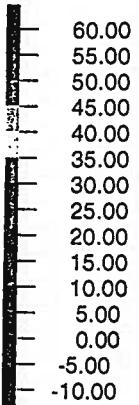


ADINA

MODE 5, TIME 4.000

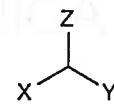
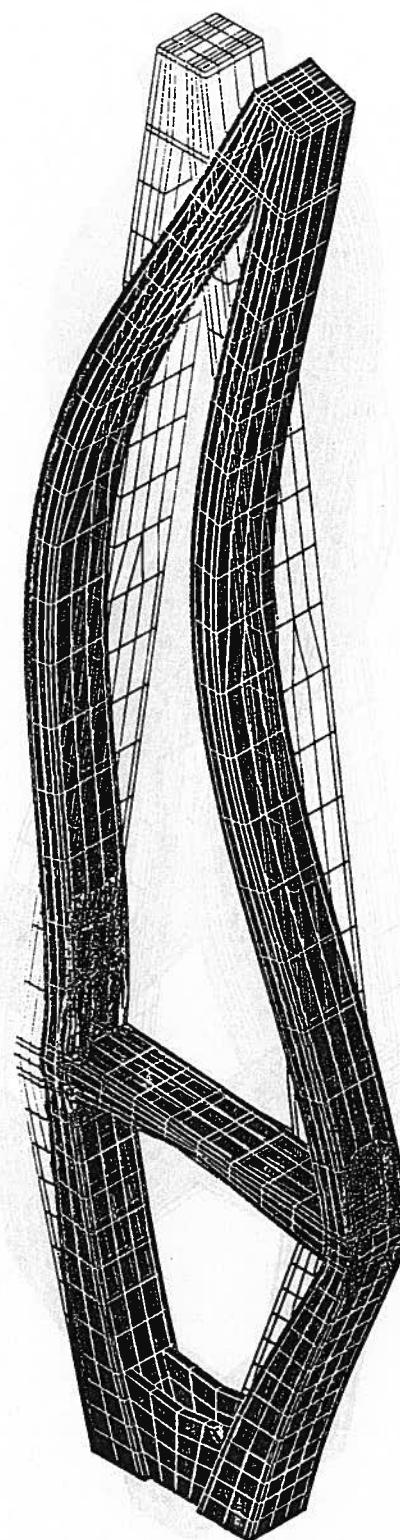


SMOOTHED
SIGMA-P1
RST CALC
MODE 5,
TIME 4.000

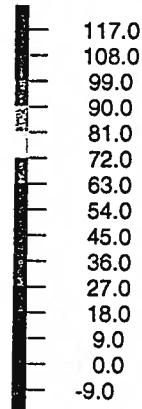


ADINA

MODE 6, TIME 4.000

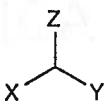


SMOOTHED
SIGMA-P1
RST CALC
MODE 6,
TIME 4.000

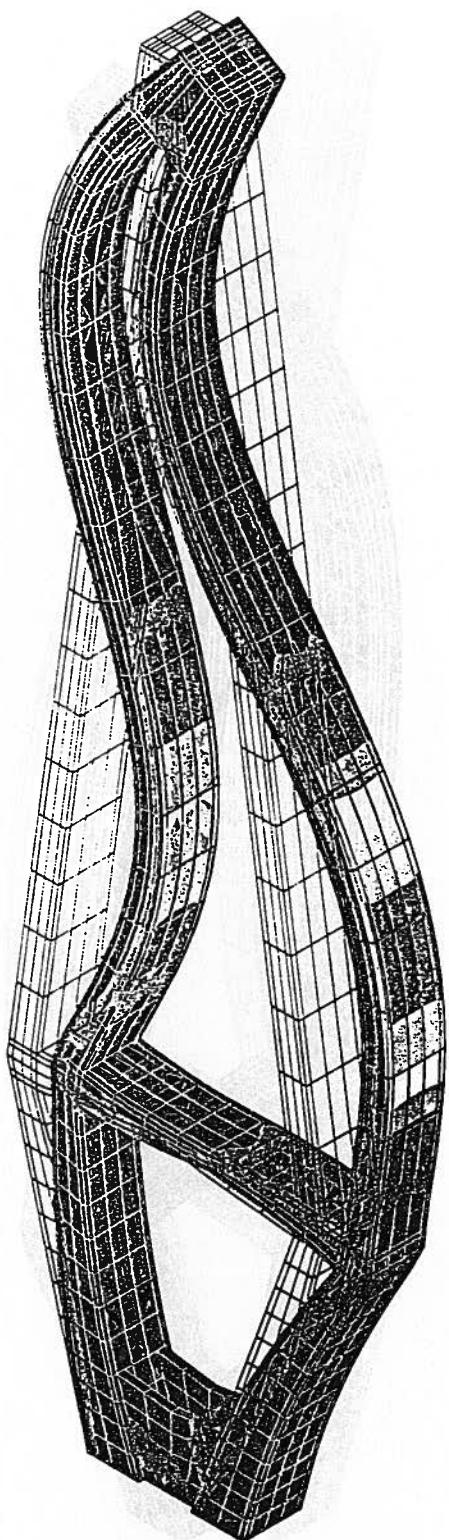
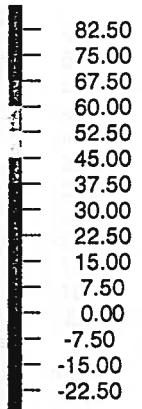


ADINA

MODE 7, TIME 4.000

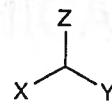


SMOOTHED
SIGMA-P1
RST CALC
MODE 7,
TIME 4.000

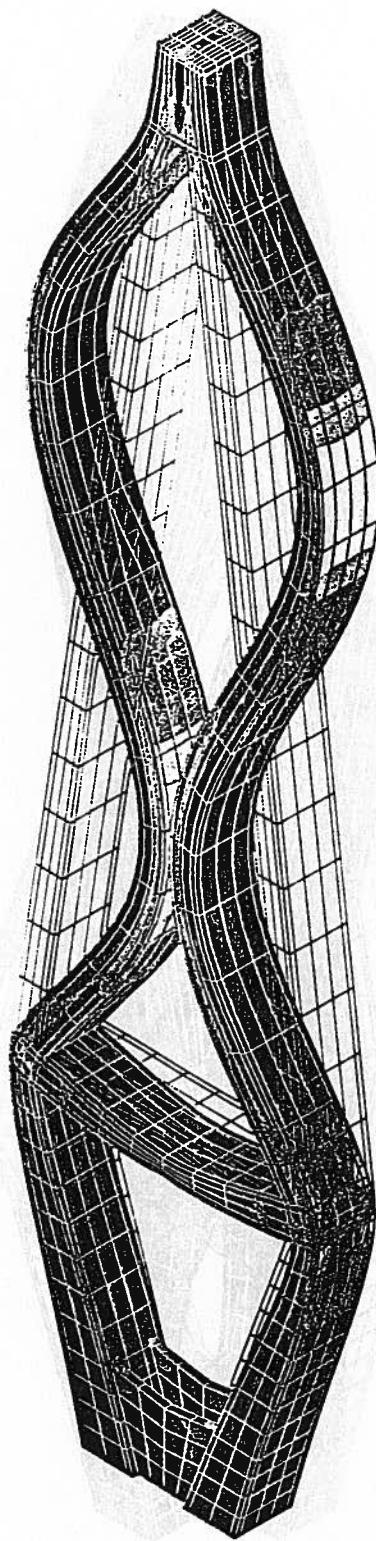
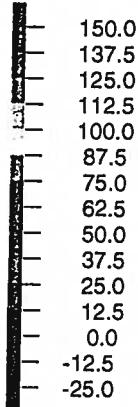


ADINA

MODE 8, TIME 4.000

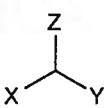
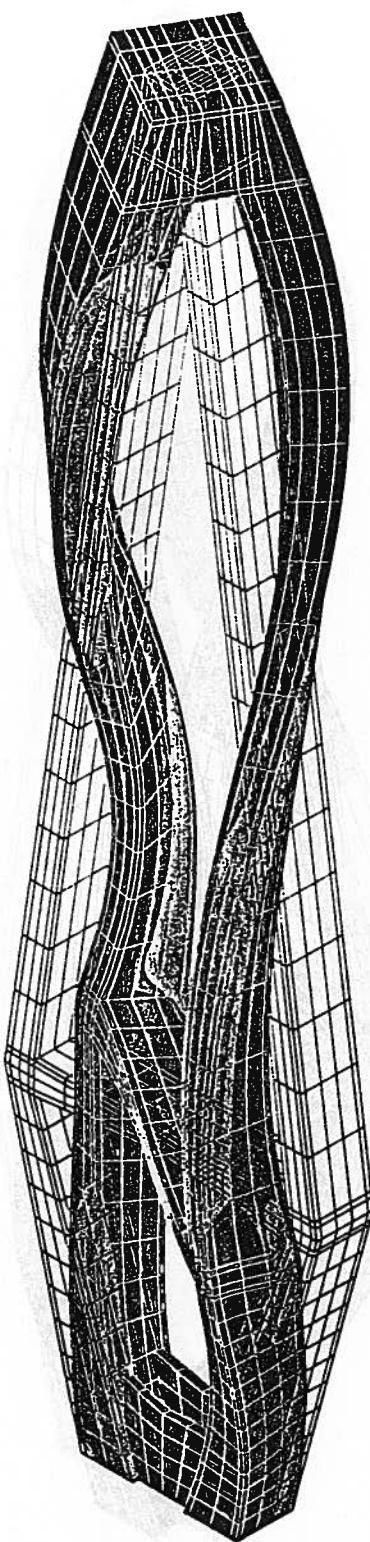


SMOOTHED
SIGMA-P1
RST CALC
MODE 8,
TIME 4.000

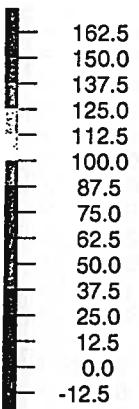


ADINA

MODE 9, TIME 4.000

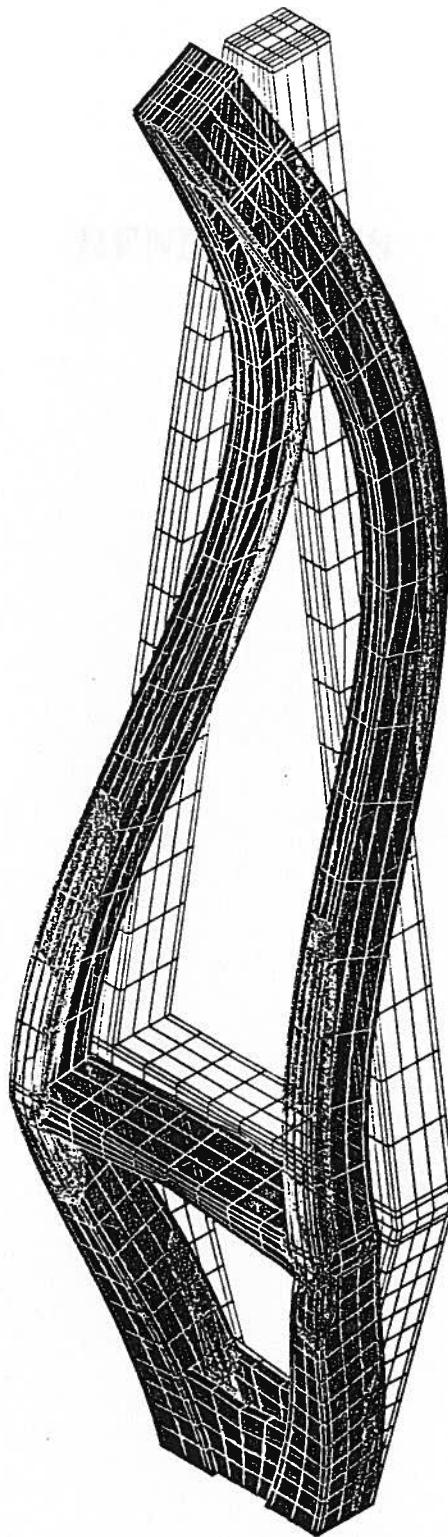
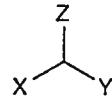


SMOOTHED
SIGMA-P1
RST CALC
MODE 9,
TIME 4.000

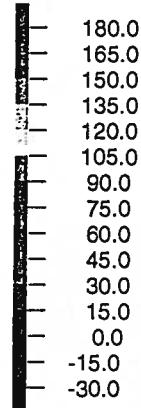


ADINA

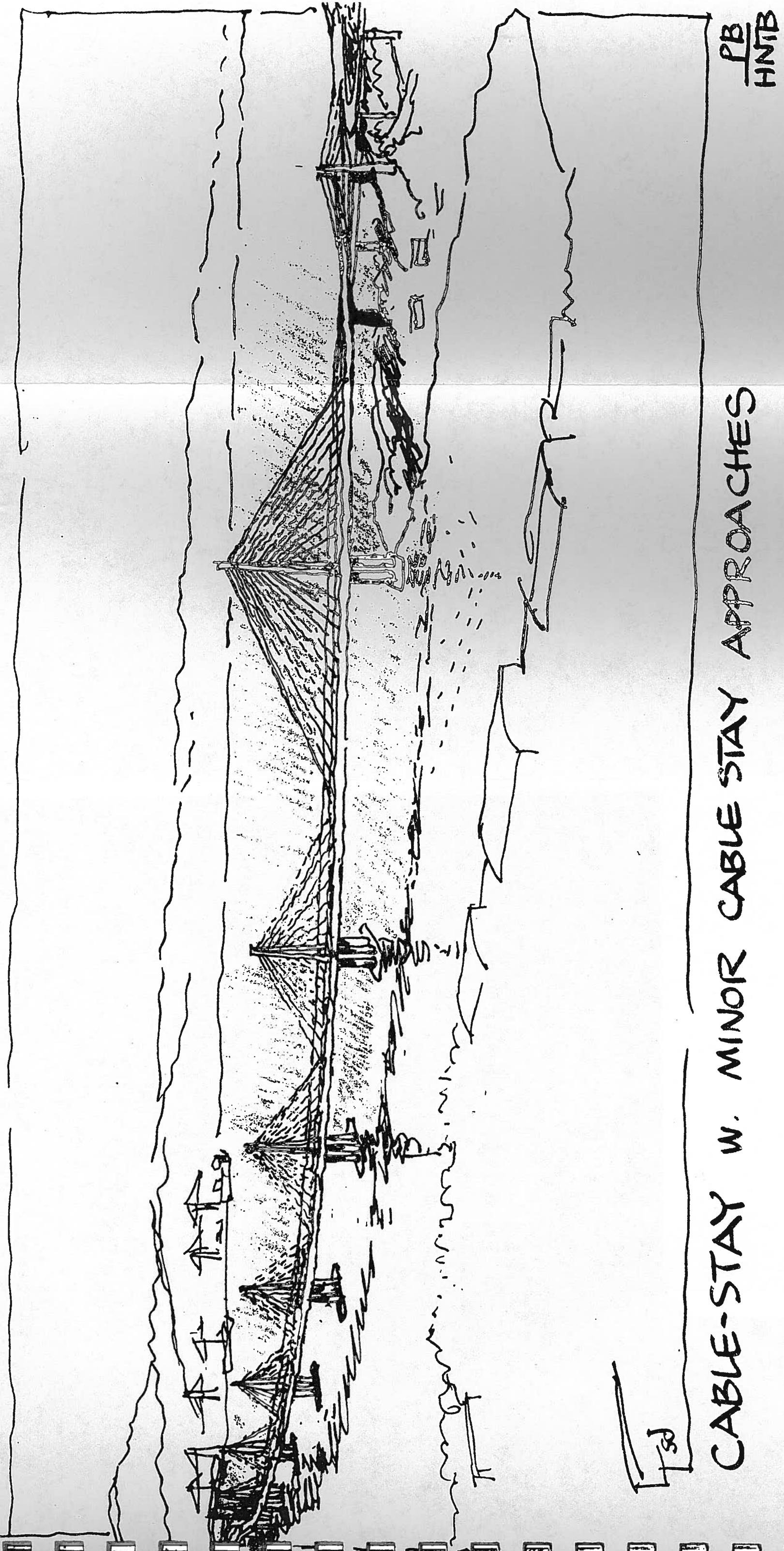
MODE 10, TIME 4.000



SMOOTHED
SIGMA-P1
RST CALC
MODE 10,
TIME 4.000



RENDERINGS

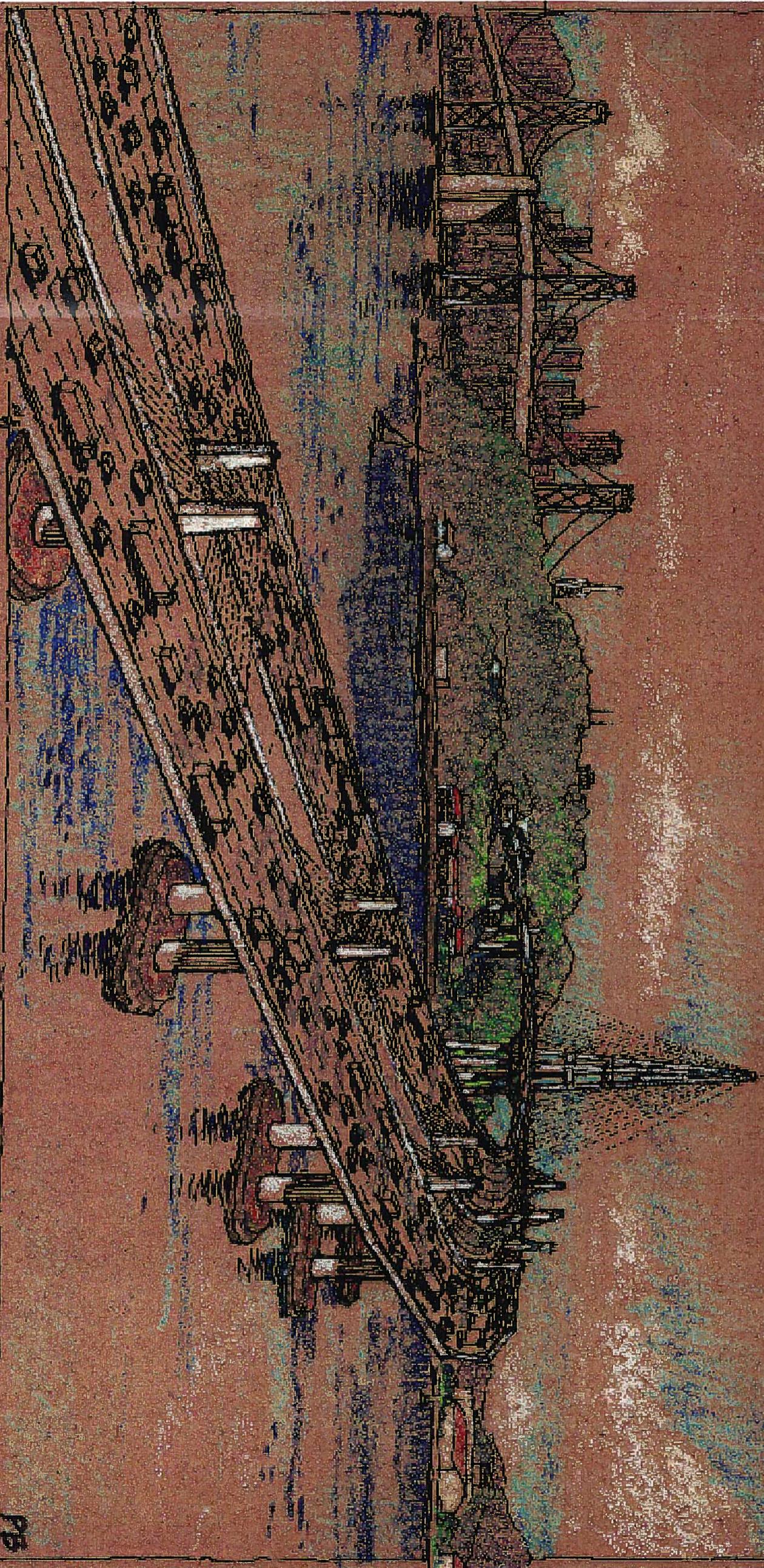


CABLE-STAY W. MINOR APPROACHES

PB
HNB

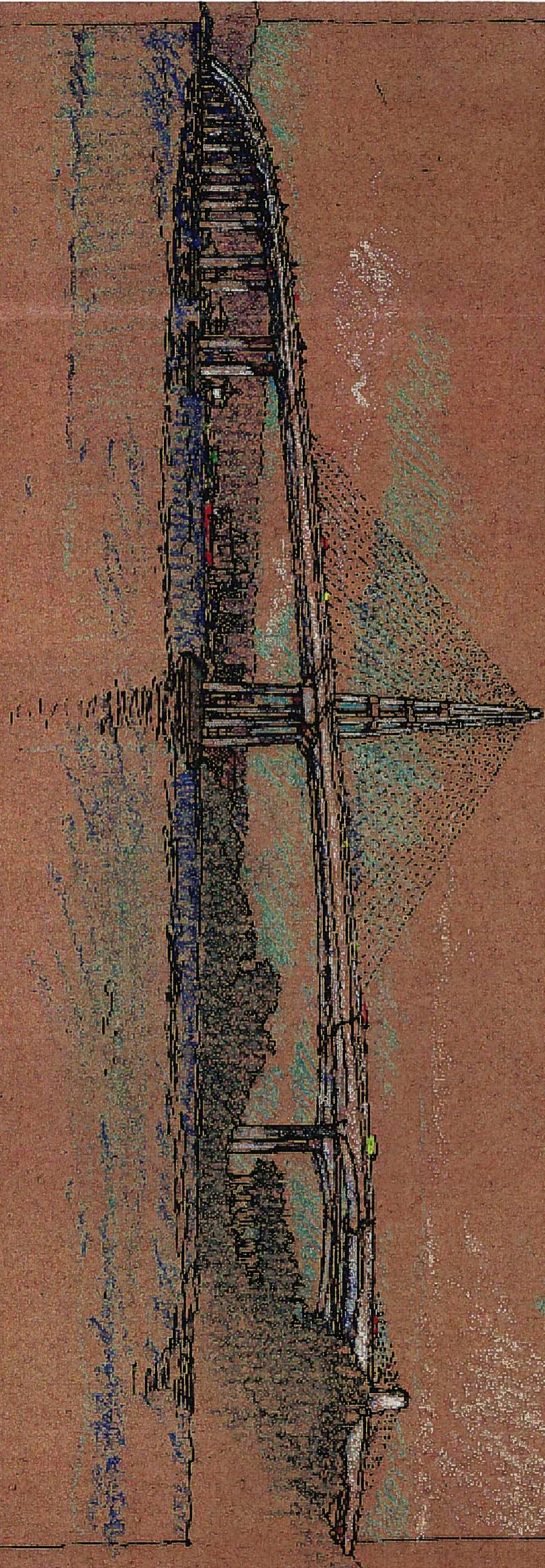
SINGLE TOWER MAIN SPAN W. EXTRAPOLATED
GIRDERS APPROACH

HHTB



SINGLE TOWER CABLE-STAY MAIN SPAN W. EXTRADOSSED
GIRDER APPROACH

PE
HNTS



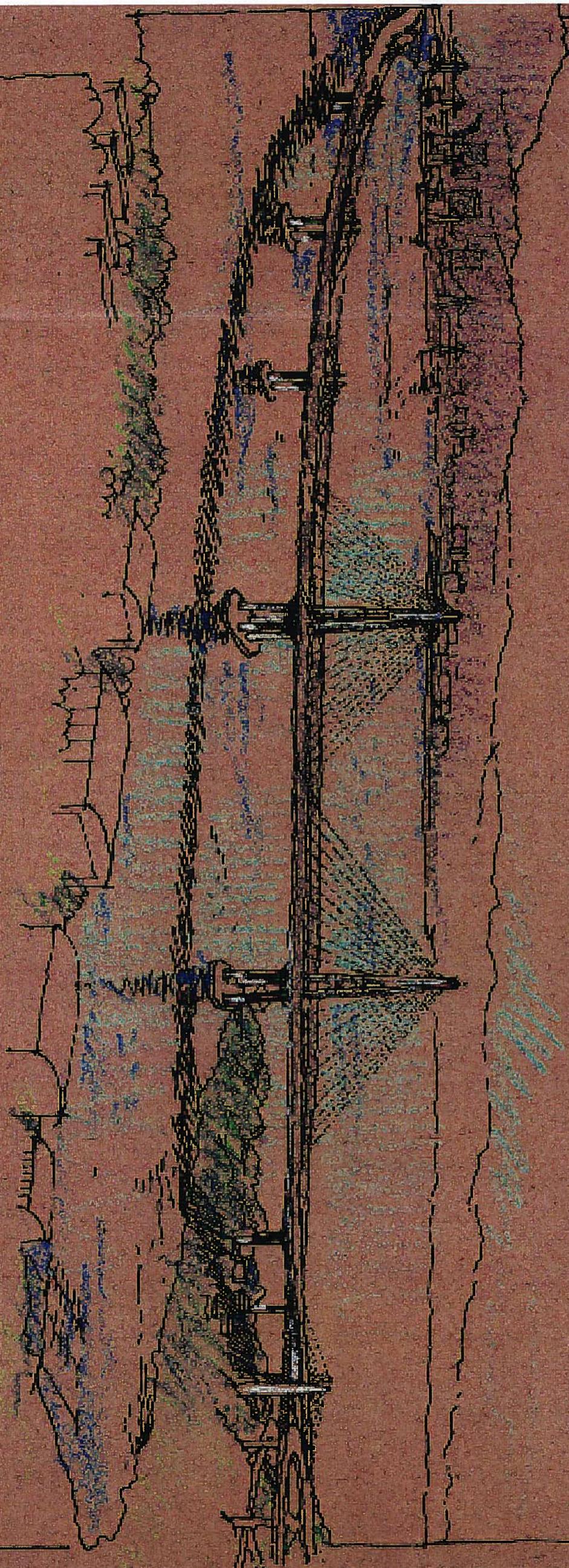
SINGLE TOWER MAIN SPAN W. EXTRADOSSED
GIRDERS APPROACH

PB
WHITE



DOUBLE TOWER CABLE-STAY MAIN SPAN W.
EXTRADOSSED GIRDER APPROACH

PP
HMTB



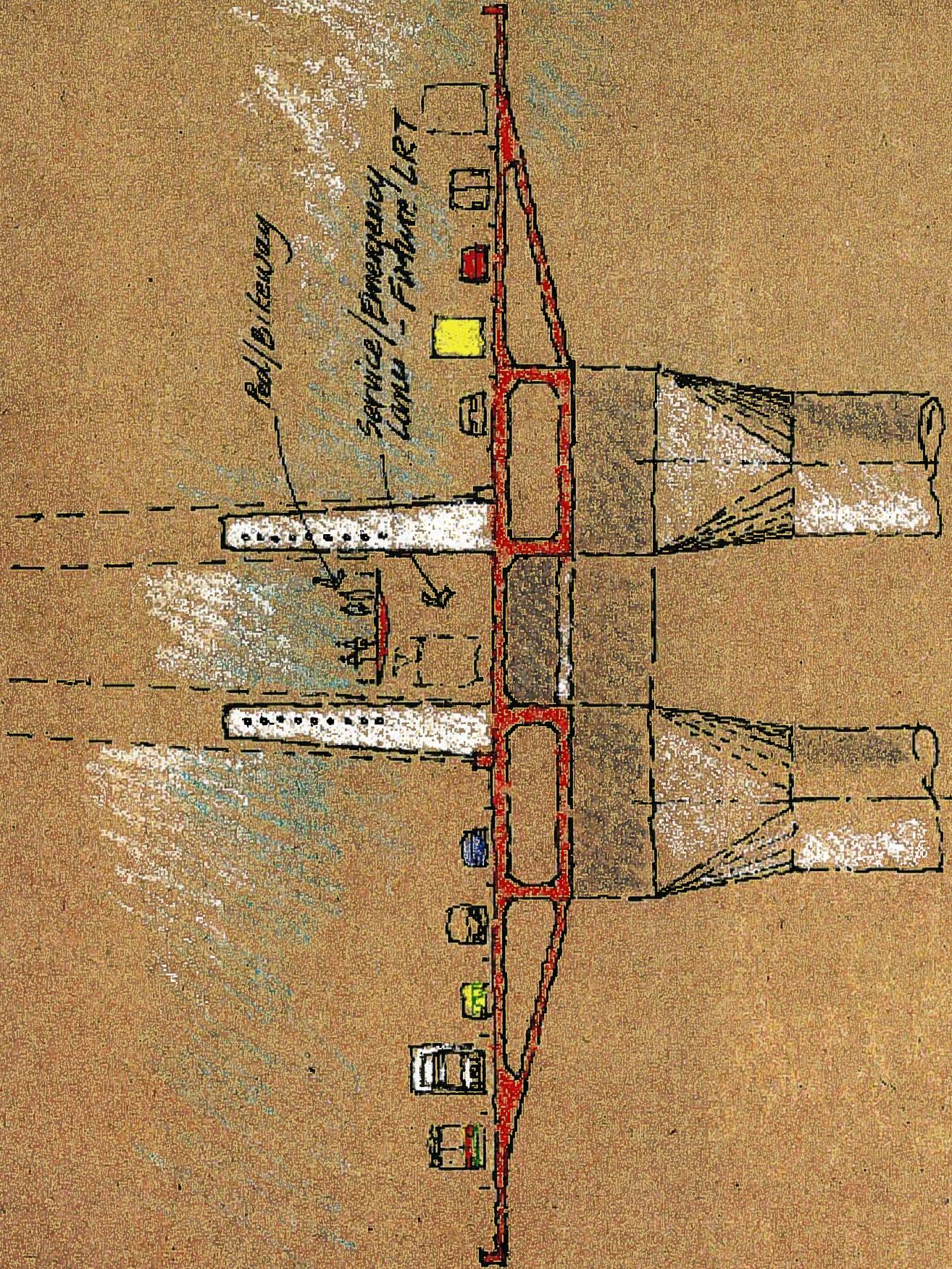
470' SEL-TOWER CABLE STAY MAIN SPAN
W. EXTRADOSSED GIRDERS APPROACH



PE
JIM

SCHEMATIC SECTION

PB
HNTB

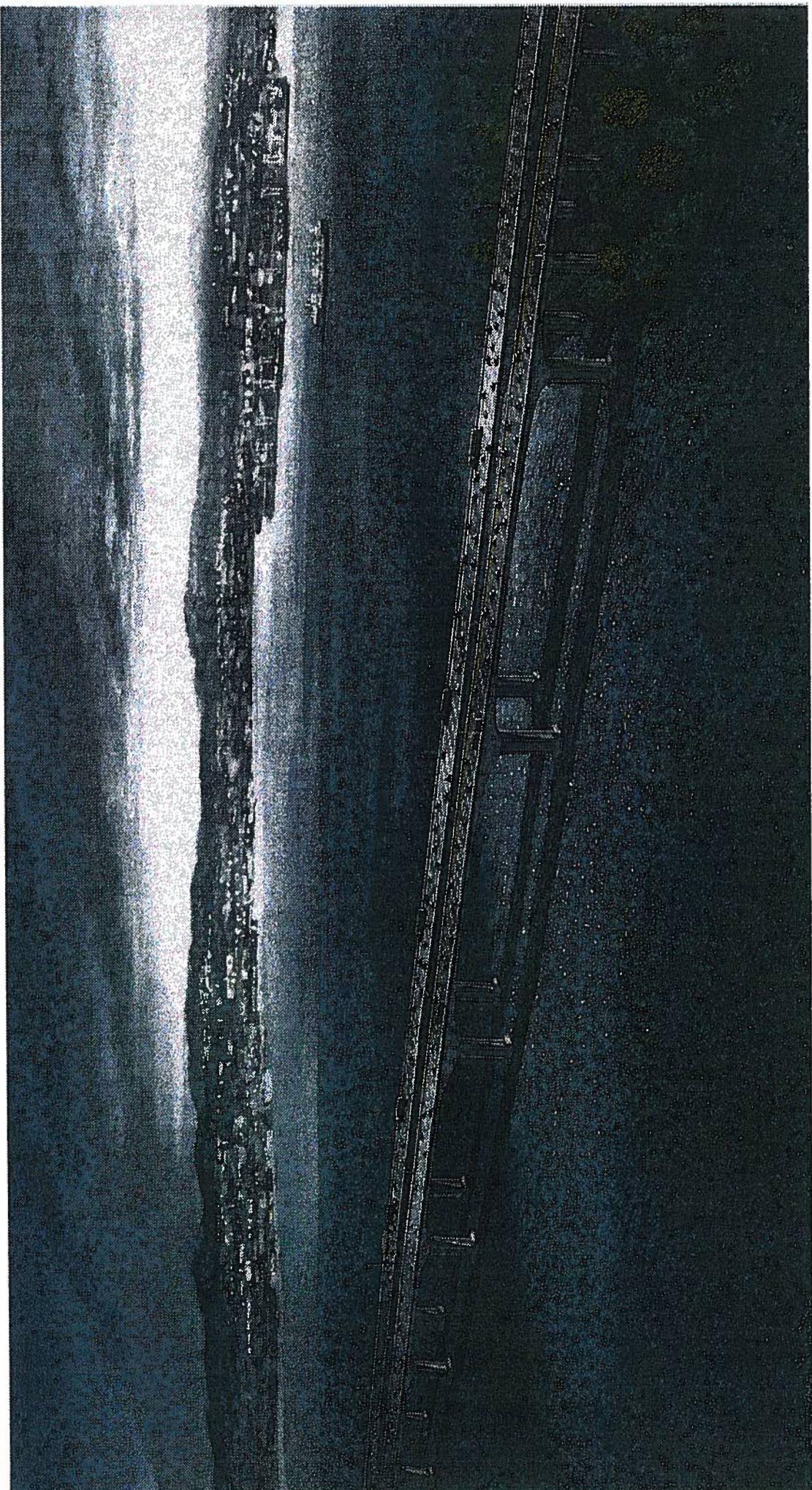


427-3

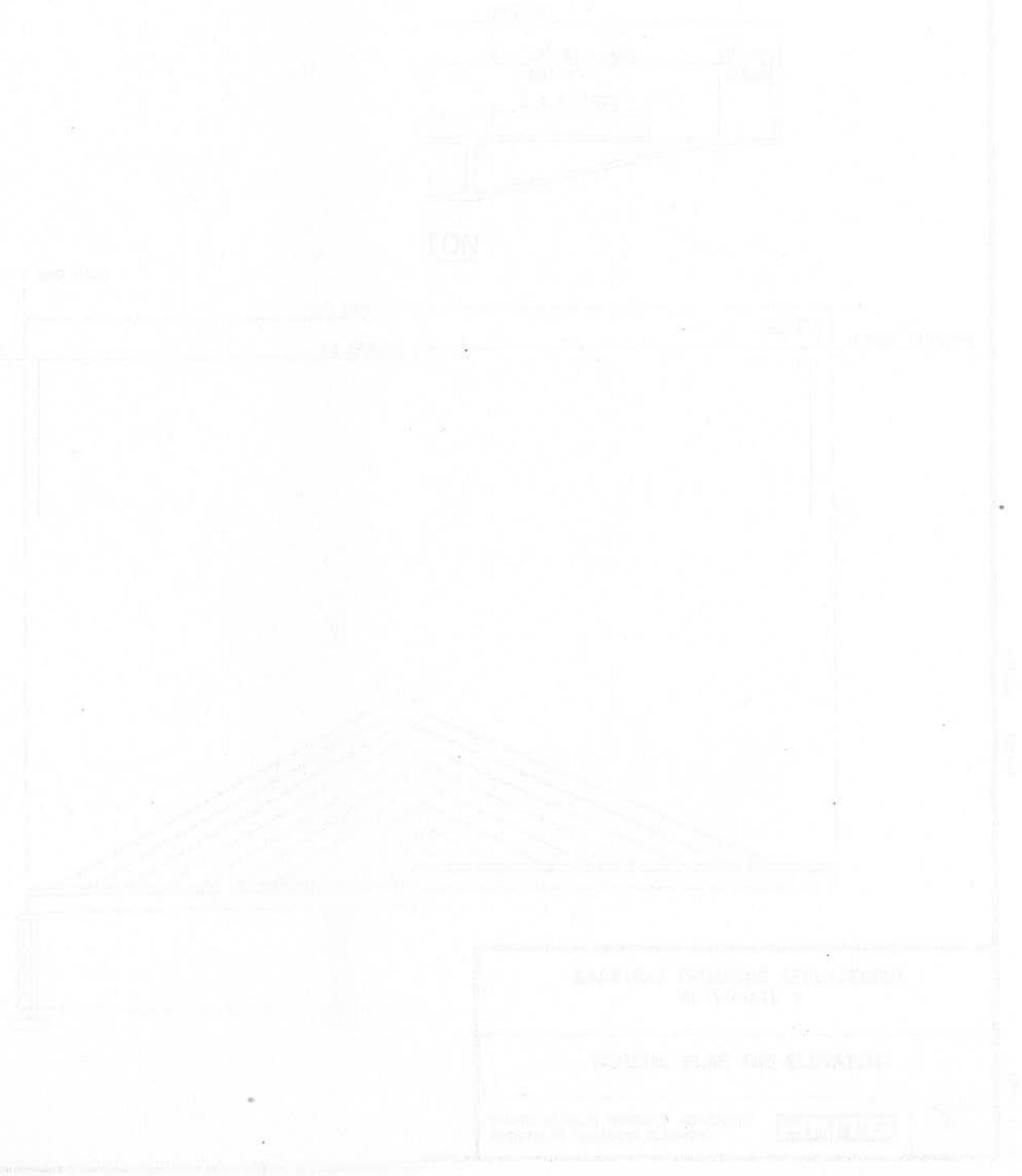
AS PRACTICALLY

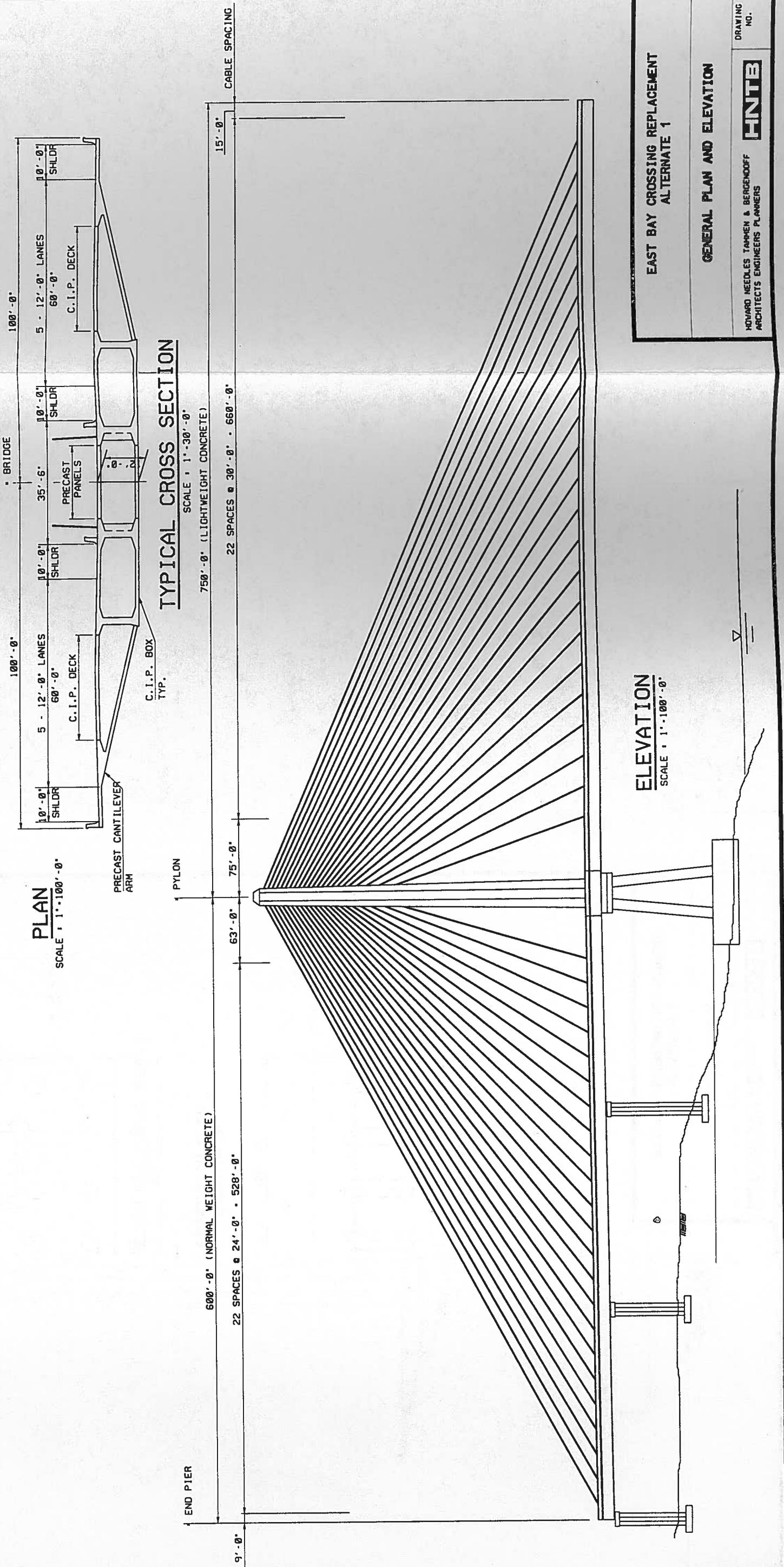
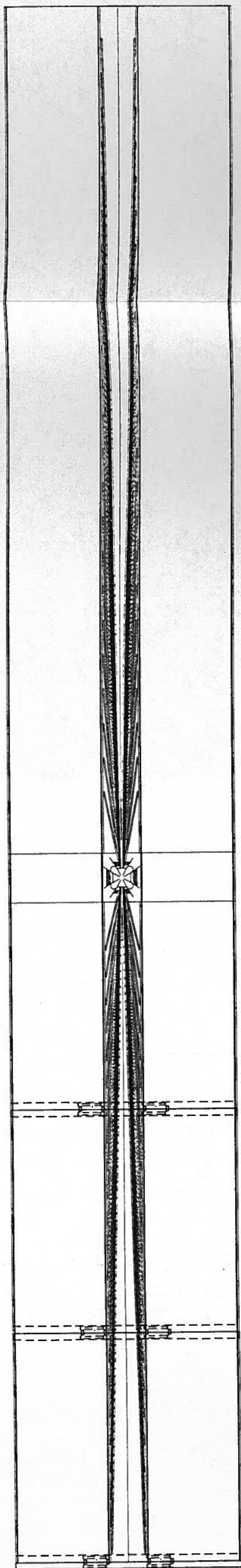


© 1980 The Estate of Edie Baskin



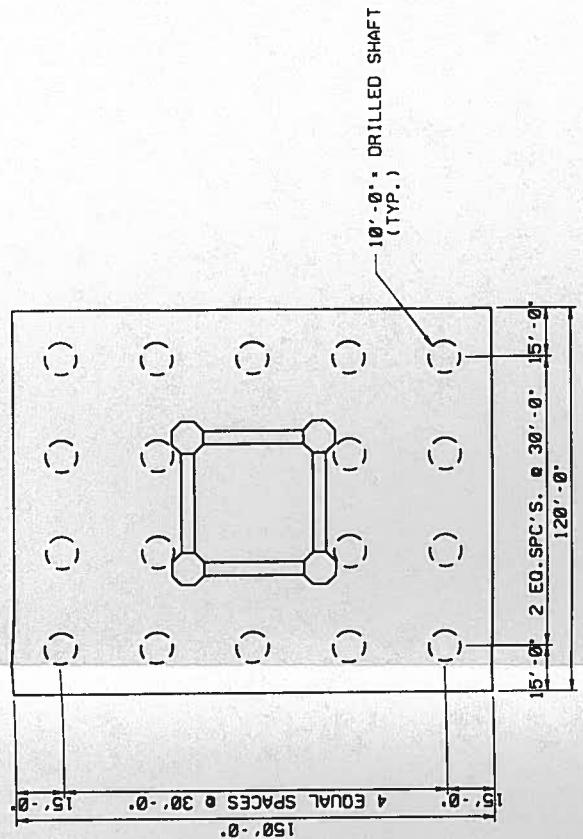
GENERAL PLANS & DETAILS





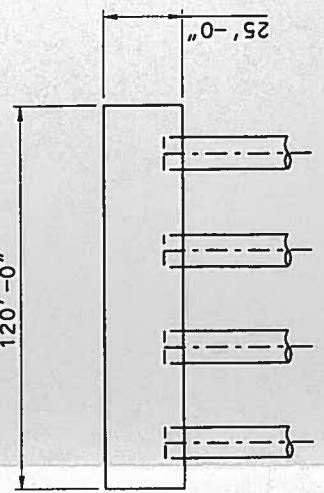
\$DATES \$TIME\$

\$8WDN 0113\$



FOUNDATION PLAN SCALE: 1'-0" - 0'.

SCALE: 1° - 30' - 0'



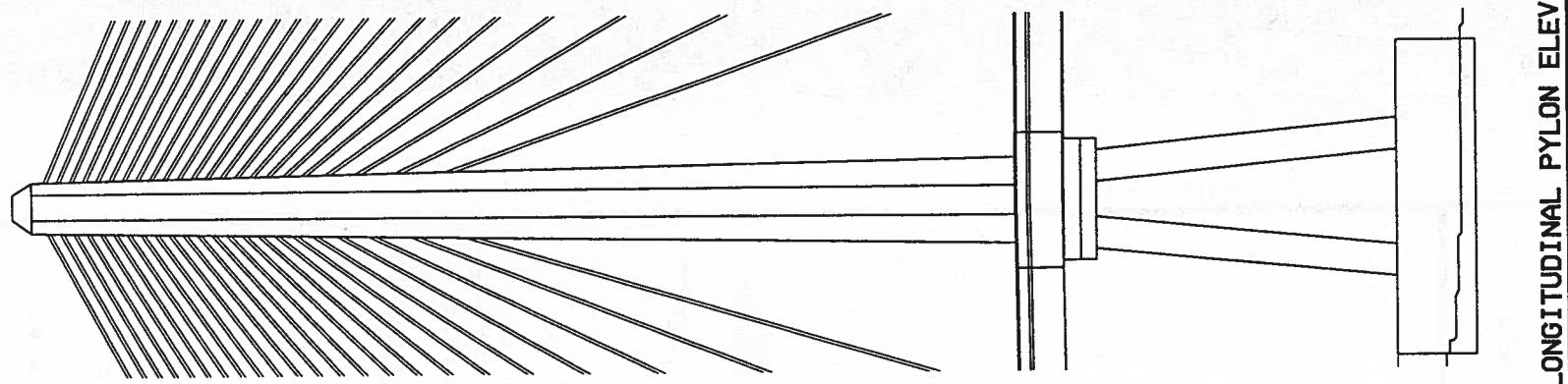
FOUNDATION ELEVATION SCALE: 1" - 30' - 0"

SCAL

**EAST BAY CROSSING REPLACEMENT
ALTERNATE 1**

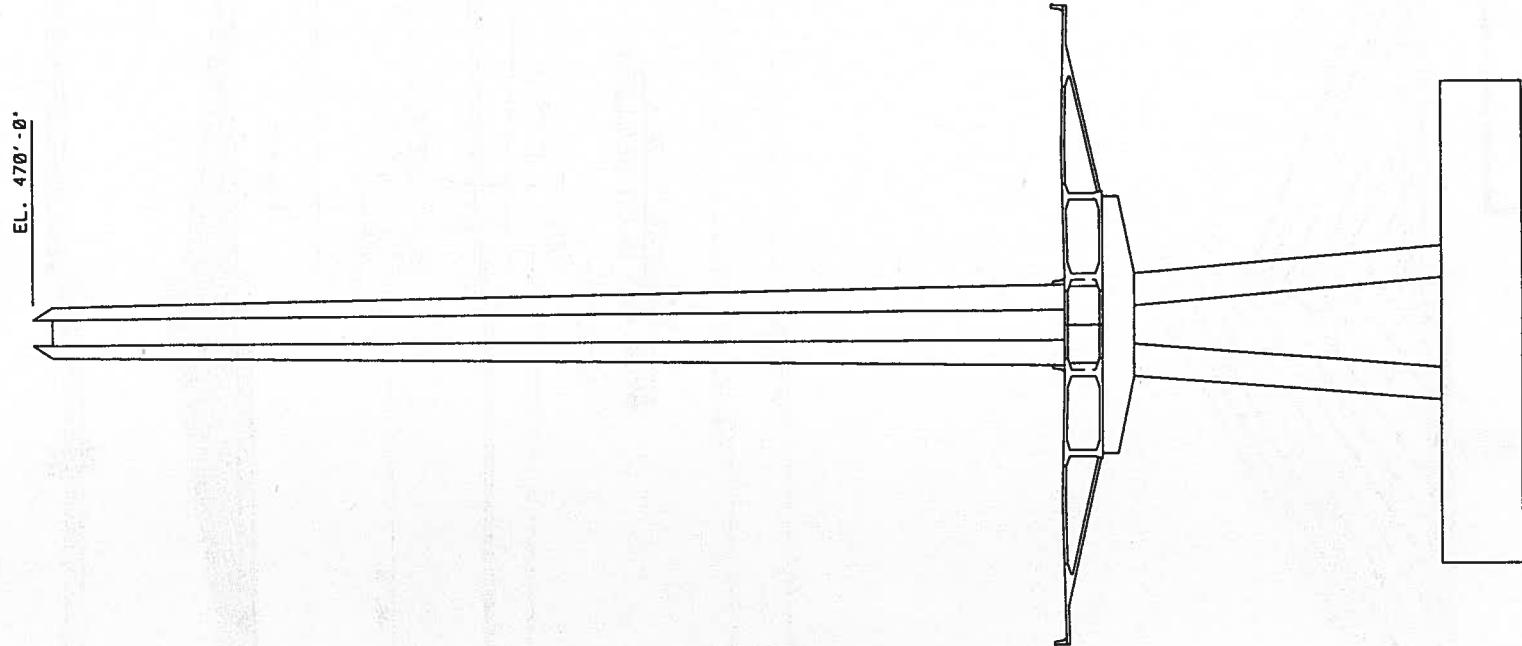
POLYMER LETTERS

HOWARD NEEDLES TAHMEN & BERGENDOFF
ARCHITECTS ENGINEERS PLANNERS



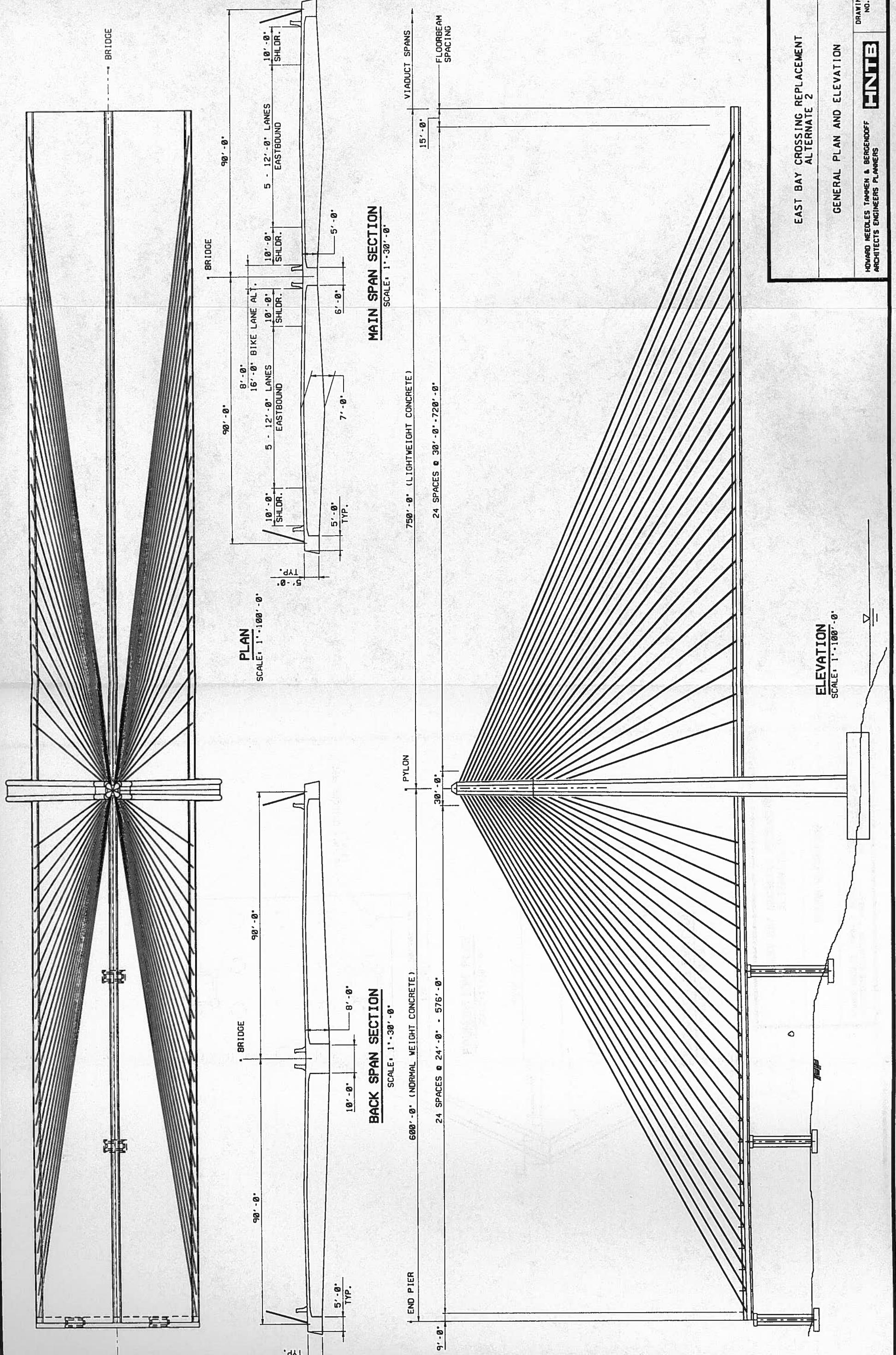
LONGITUDINAL PYLON ELEVATION

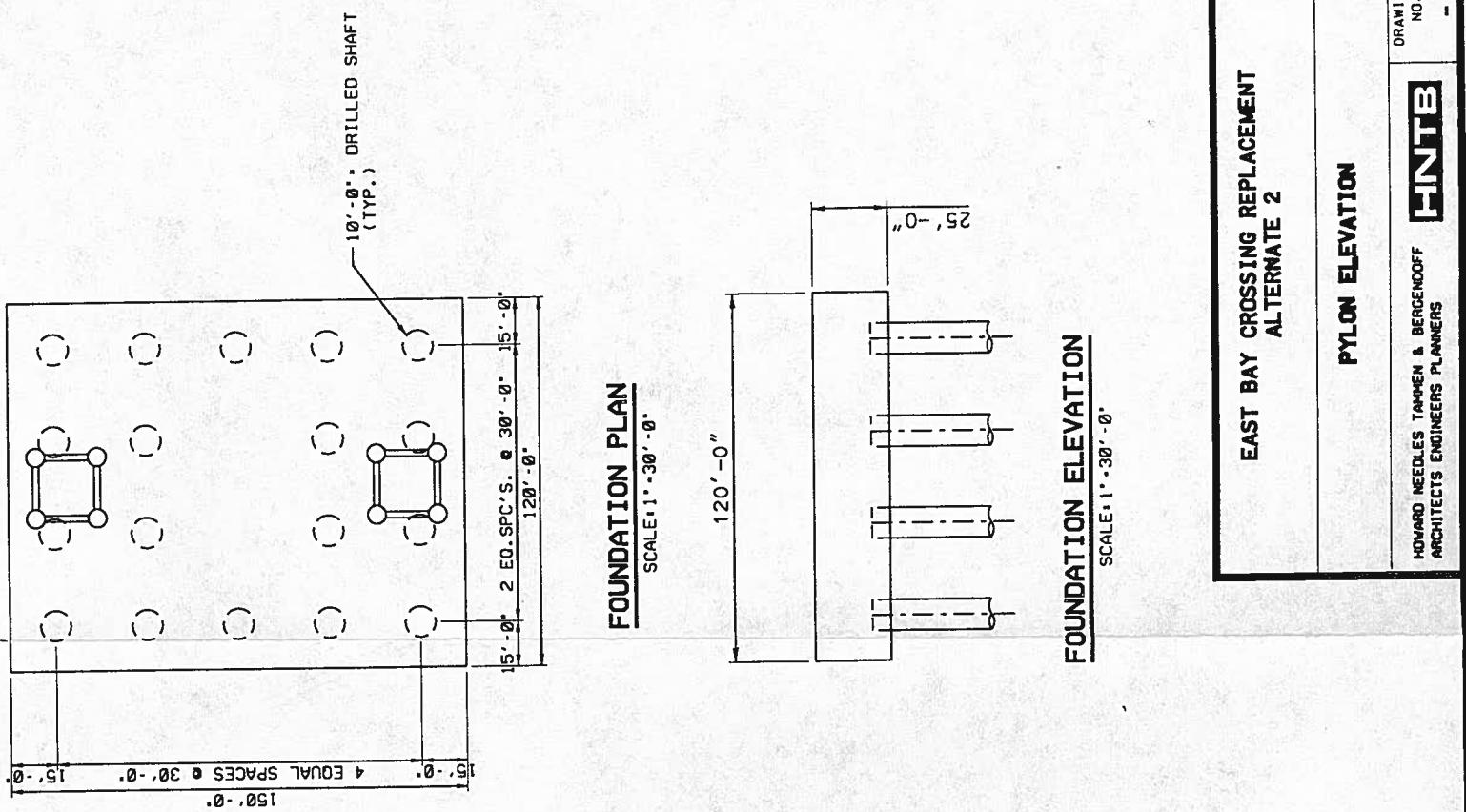
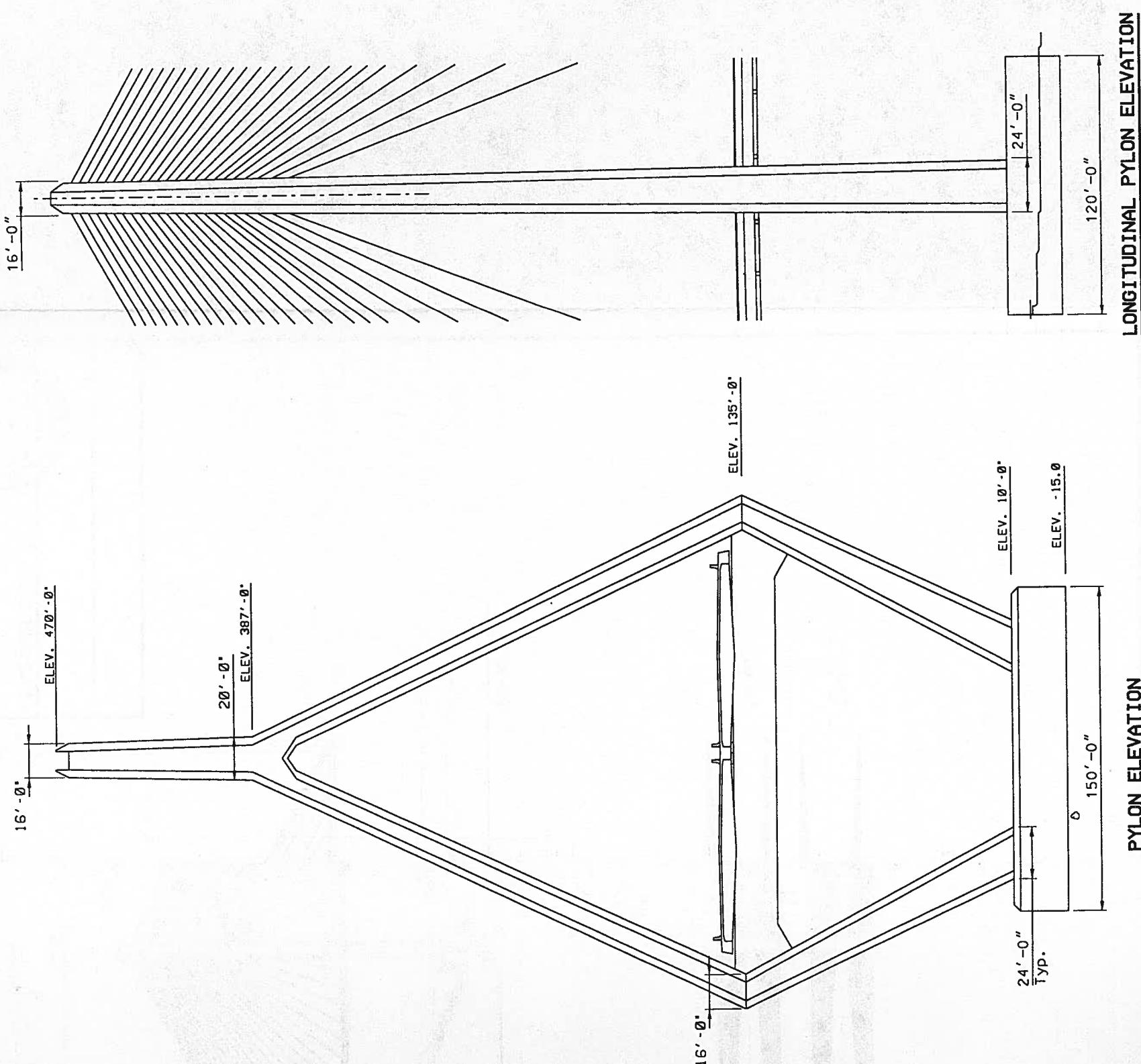
SCALE: 1' = 30' - 0'.



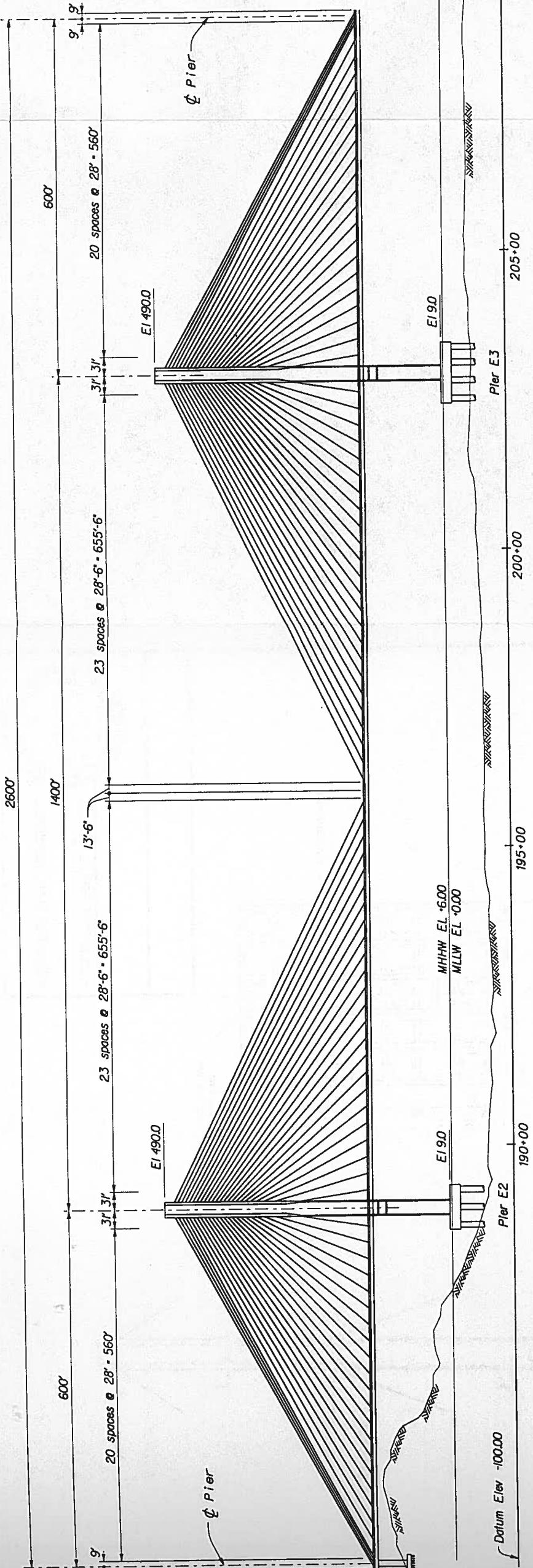
PYLON ELEVATION
SCALE 1 : 300 - 0.

SCALE: 1" = 30' - 0"

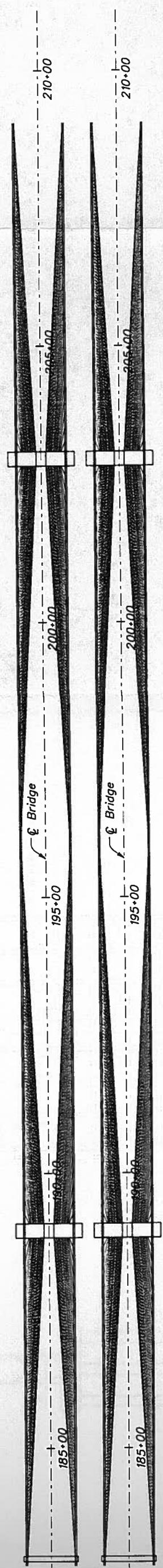




DIST	COUNTY	ROUTE	TOTAL PROJECT NO.	SHEETS
PROFESSIONAL ENGINEER REGISTERED CIVIL ENGINEER No. _____ REGISTRATION NO. _____ CIVIL STATE OF CALIFORNIA DP. _____				
PLANS APPROVAL DATE				



ELEVATION
r-100'



PLAN
r-100'

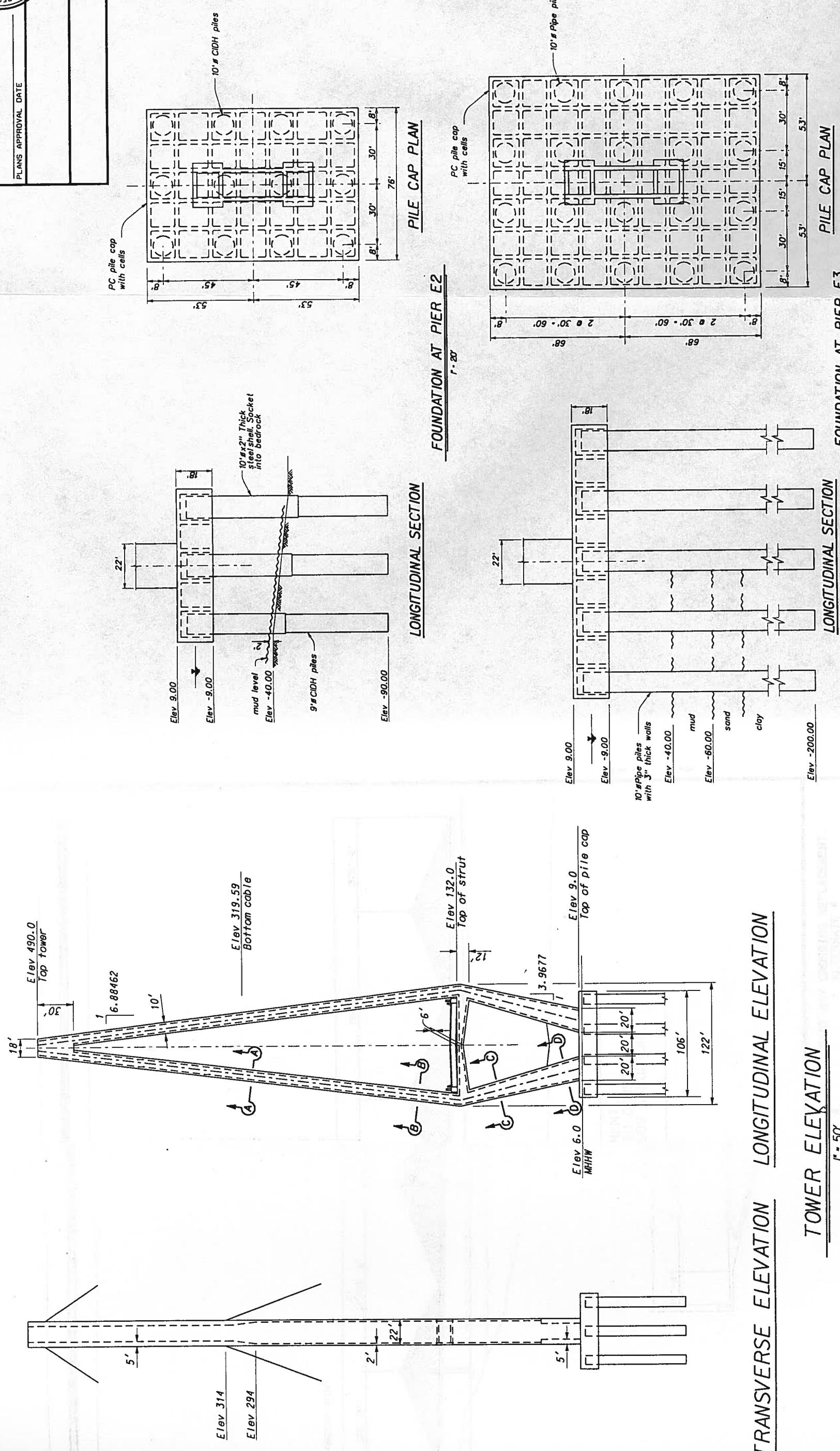
		PREPARED FOR THE		MAIN SPANS - ALTERNATE 3	
		STATE OF CALIFORNIA		DOUBLE-DIAMOND PYLON-THREE SPAN CABLE STAY	
		DEPARTMENT OF TRANSPORTATION		REVISION DATES (Preliminary Stage Only)	
DESIGN DETAILS	CHECKED	PROJECT ENGINEER	POST MILE	DISREGARD PRINTS BEARING EARLIER REVISION DATES	NUMBER OF SHEETS
QUANTITIES	INCHES	NAME	POST MILE	FOR REDUCED PLANS	STORY DRAFT DATE
1	2	3	CU EA	6	

A circular seal for a professional engineer. The outer ring contains the words "REGISTERED PROFESSIONAL ENGINEER" at the top and "CALIFORNIA STATE OF" at the bottom. The inner circle contains "CIVIL" at the top, "EXP." in the center, and "No." at the bottom.

REGISTERED CIVIL ENGINEER

B1 AND APPROVAL DATE

1ST COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS

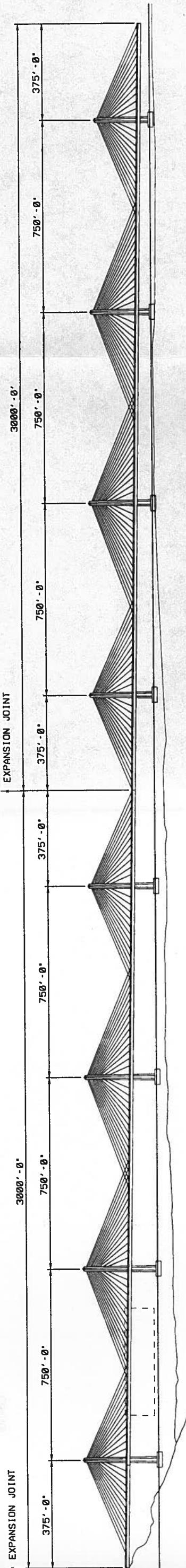


ALTERNATIVE 3

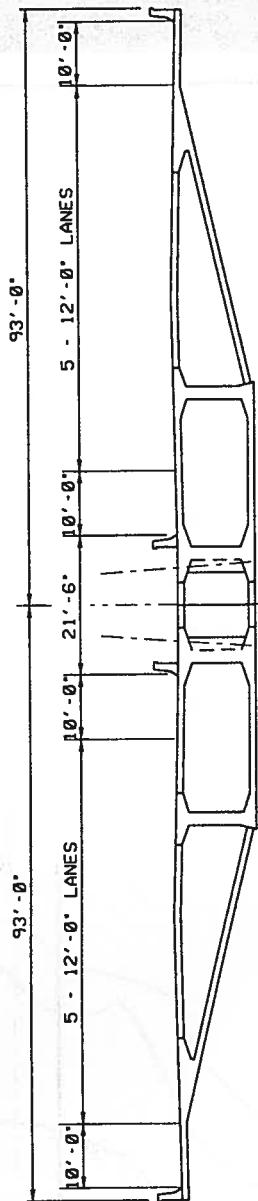
PROJECT ENGINEER
PREPARED FOR THE
STATE OF CALIFORNIA
EDWARD G. BROWN

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION		PROJECT ENGINEER	
CHEKED	SITE NO.	POST MILE	DISREGARD PRINTS BEARING EARLIER REVISION DATES →
CHEKED			REVISION DATES (PRELIMINARY STAGE ONLY)
CHEKED			SHEET OF
	ORIGINAL SCALE IN INCHES FOR REDUCED PLANS	CU	
	6	1	
	1	2	
	3	3	
	EA	EA	

PLAN
SCALE : 1'-400'-0"

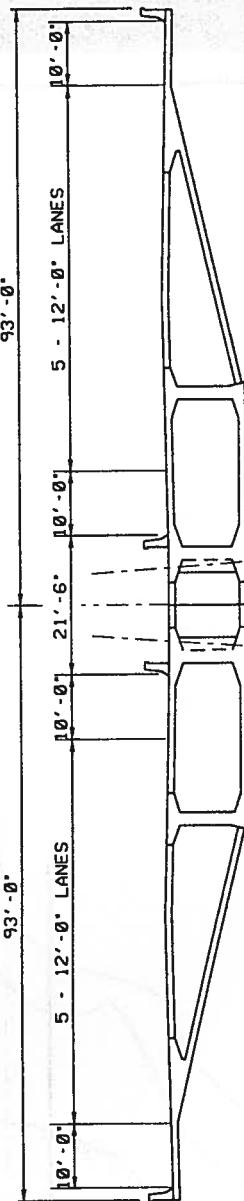


ELEVATION
SCALE : 1'-400'-0"



THIS CONCEPT IS CURRENTLY
UNDER DEVELOPMENT TO CONSIDER
THE USE OF AN EXTRADOS STRUCTURE
IN THE APPROACH

TYPICAL CROSS SECTION
SCALE : 1'-30'-0"



EAST BAY CROSSING REPLACEMENT
ALTERNATE 4

GENERAL PLAN AND ELEVATION

HORN NEEDLES TAPPEN & BERGENDOFF
ARCHITECTS ENGINEERS PLANNERS

DRAWING
NO. -
INTB

SP116 NORMS

A circular seal for a professional engineer. The outer ring contains the text "REGISTERED PROFESSIONAL ENGINEER" at the top and "STATE OF CALIFORNIA" at the bottom. The center of the seal features three vertical lines, with "M." above the left line and "D." above the right line, separated by a horizontal line.

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

22/3

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४८६

~~C~~ Pier

—C Pier

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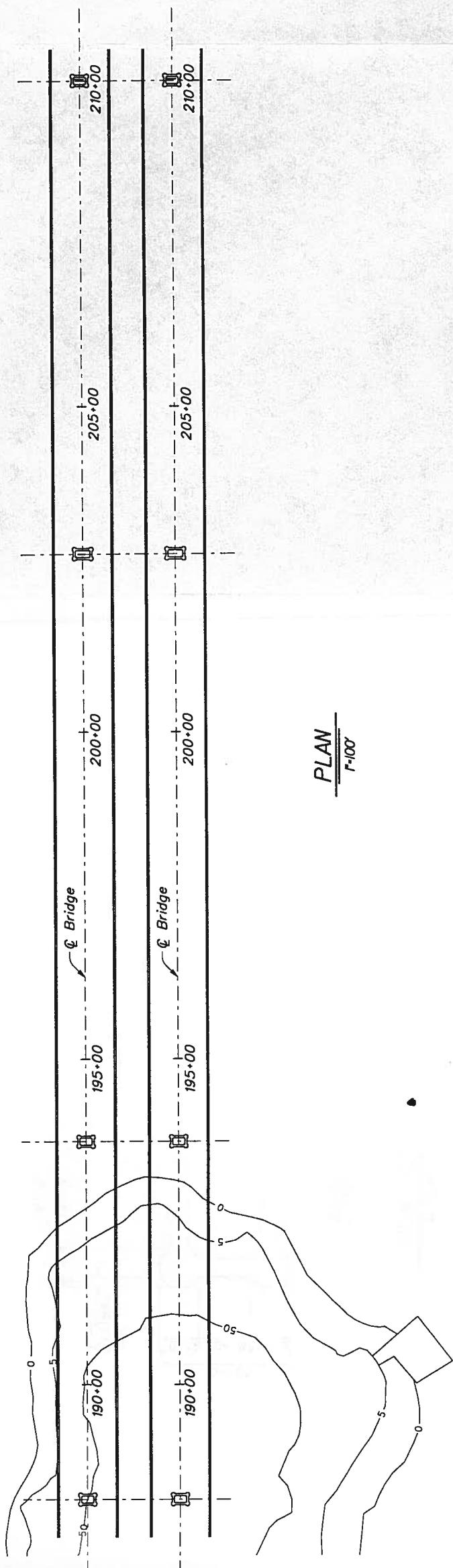
210

775

PLANS APPROVAL DATE

A cross-sectional diagram of a bridge foundation. The vertical axis represents elevation, with labels at Datum Elev -100.00, 190-00, 195-00, 200-00, 205-00, 210-00, and Elev 9.0. The horizontal axis represents distance. The diagram shows three piers labeled Pier E2, Pier E3, and Pier E4. Pier E2 is at the bottom, Pier E3 is in the middle, and Pier E4 is at the top. Each pier has multiple foundation piles extending downwards. The water level is indicated by a dashed line at Elev 9.0. The water flows from left to right, as shown by arrows. The diagram also shows the MHHW EL +6.00 and MLLW EL +0.00 levels.

ELEVATION
 $r=100$

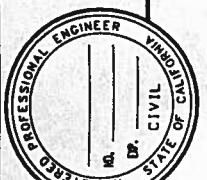


PLAN
 $r=100'$

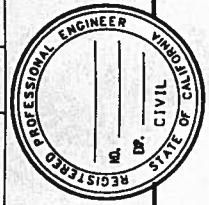
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO. TOTAL SHEETS

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE



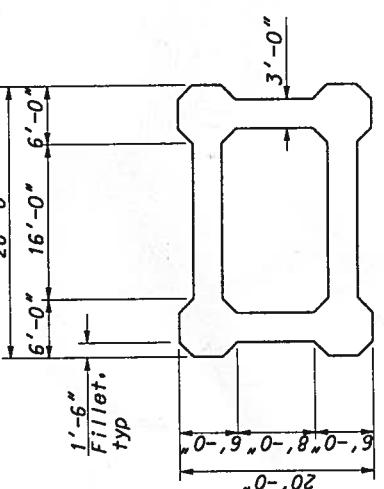
The seal is circular with the words "PROFESSIONAL ENGINEER" at the top and "REGISTERED" at the bottom. It features a central vertical line with two horizontal lines extending from it. The outer ring contains "STATE OF CALIFORNIA" at the top and "CIVIL" at the bottom.



REGISTERED CIVIL ENGINEER

POLICE APPROVAL DATE

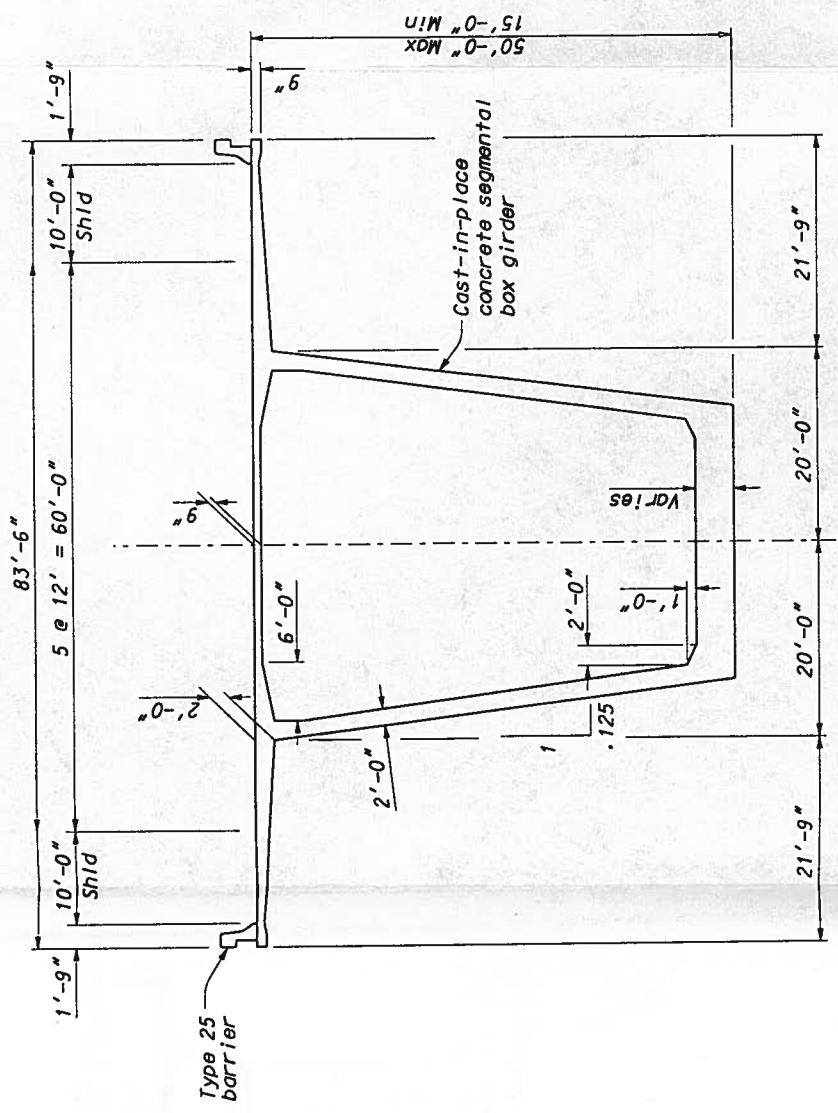
SECTION



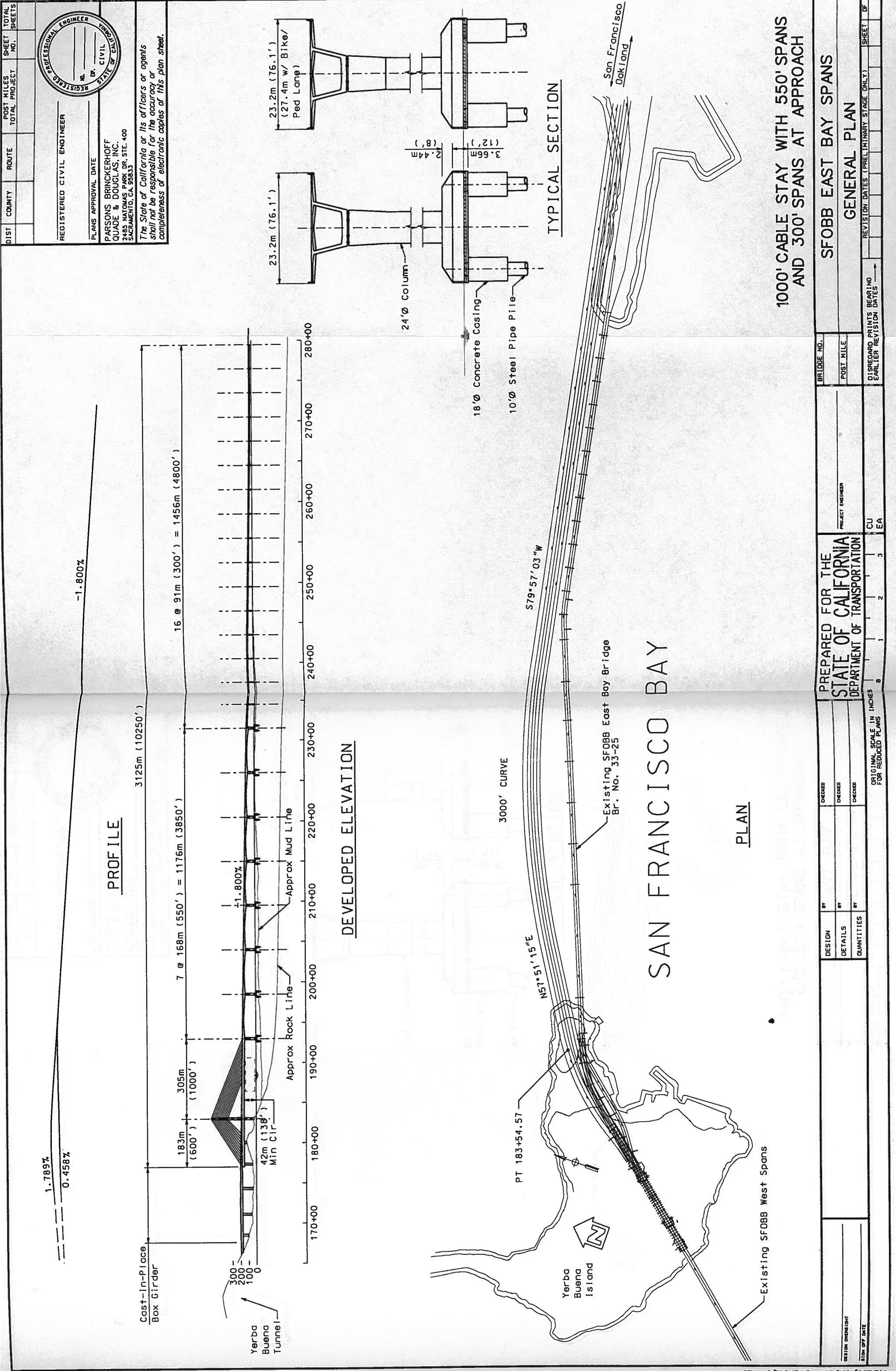
PLAW

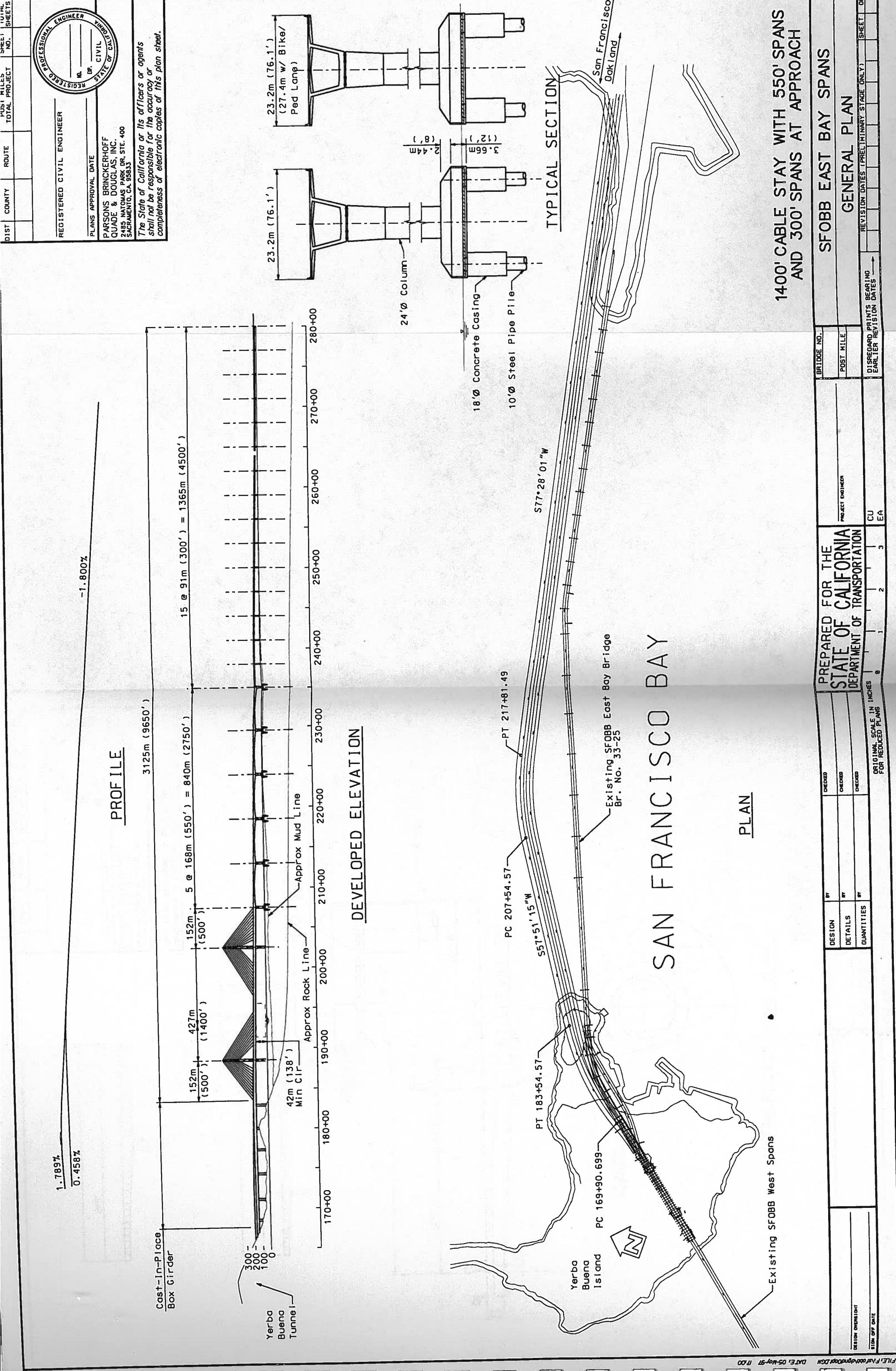
BENT E2
No Scale

TYPICAL SECTION



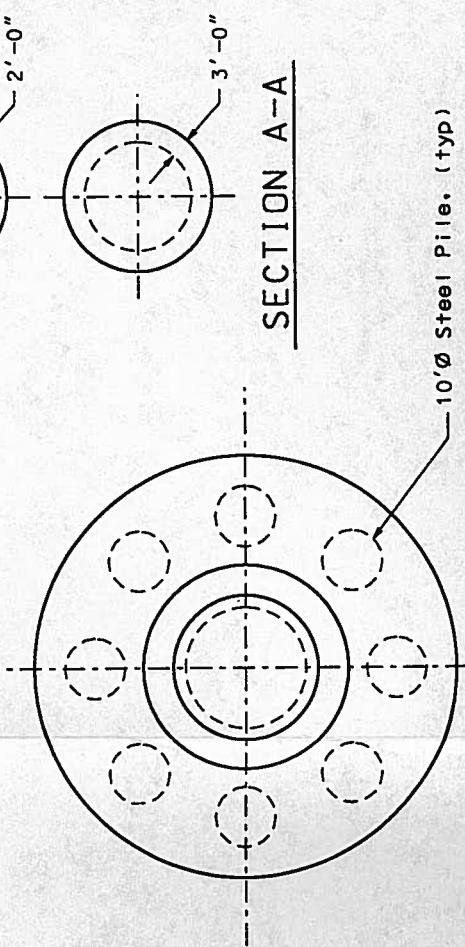
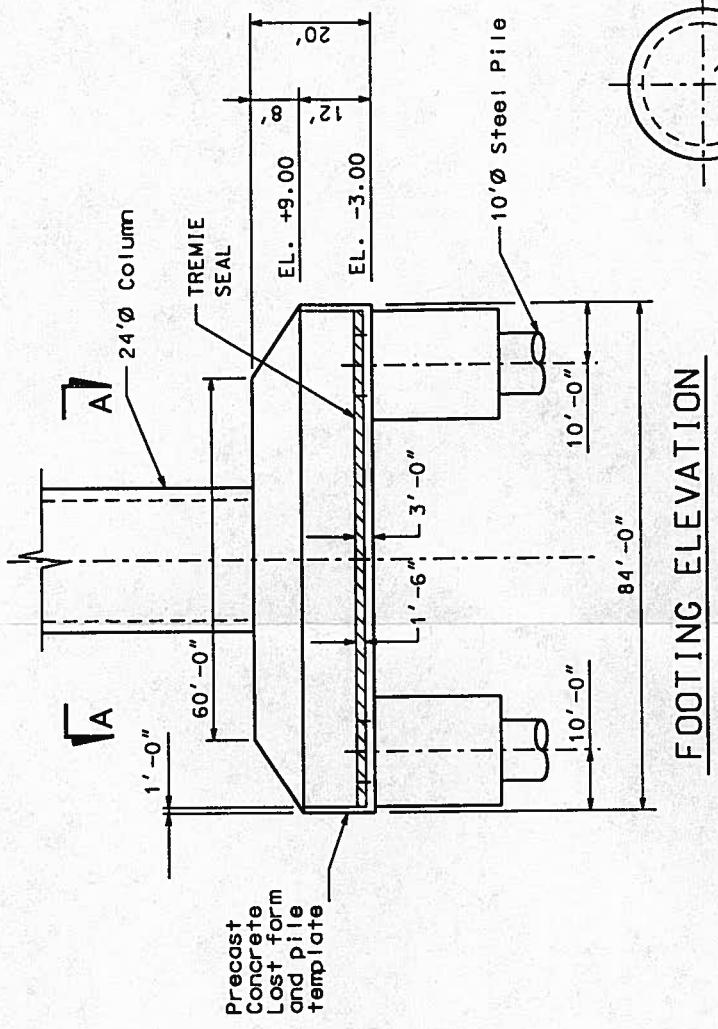
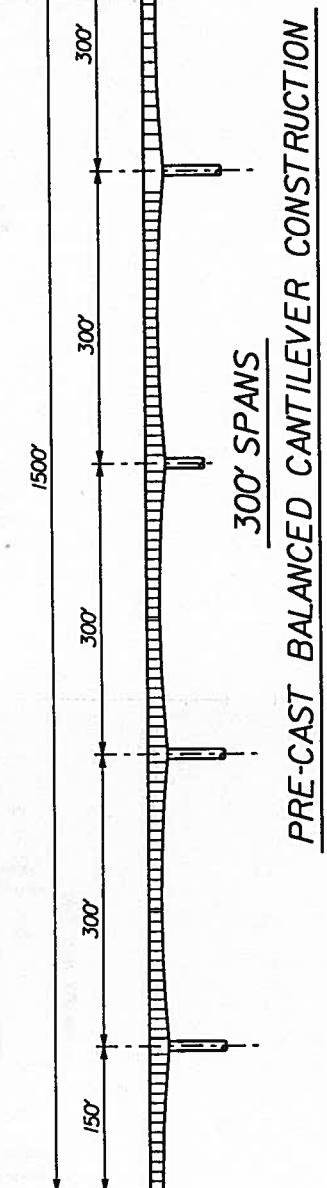
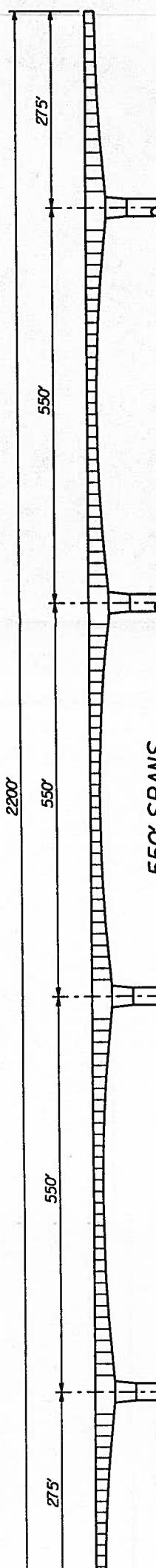
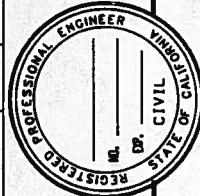
ALTERNATIVE 5



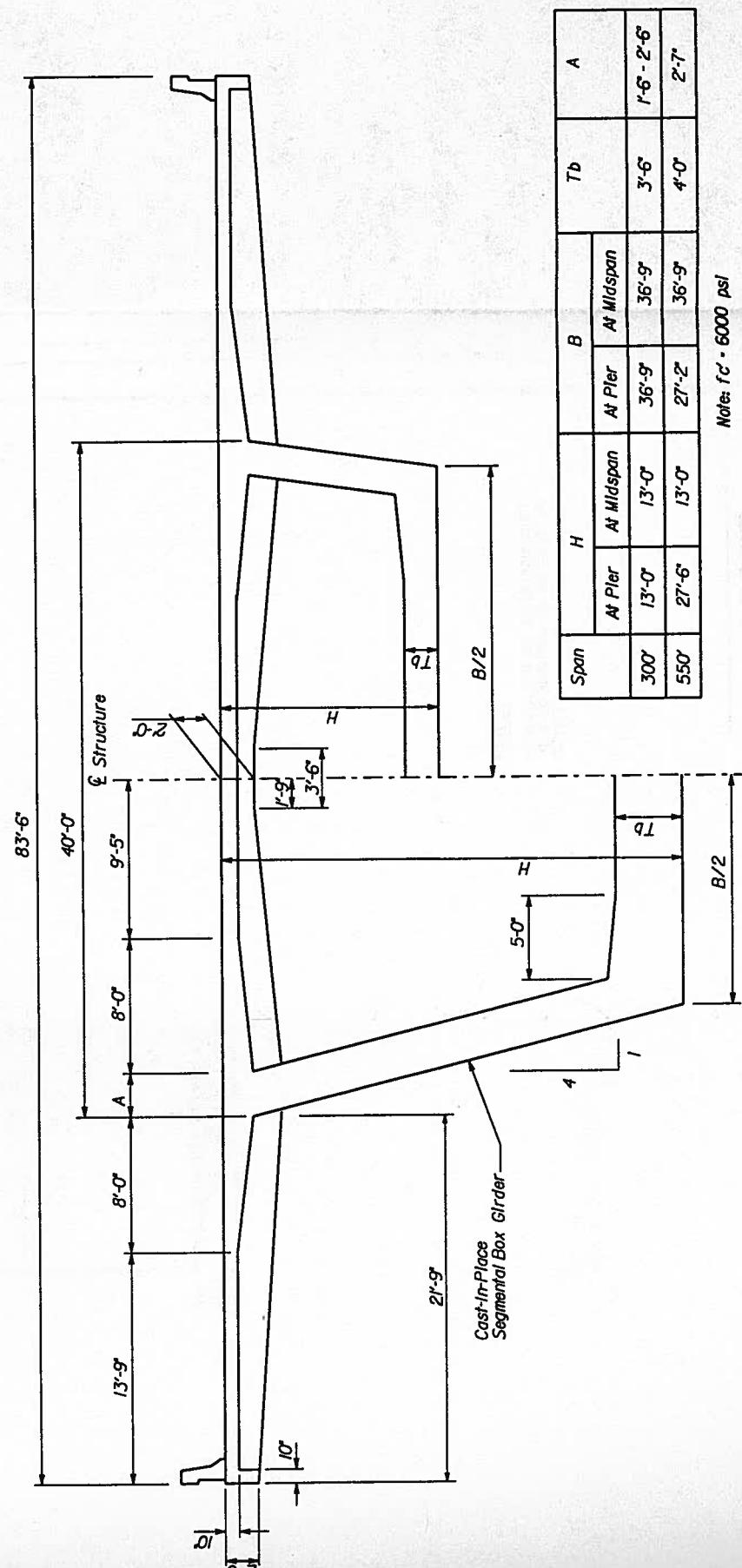


DIST	COUNTY	ROUTE	POST MILES	TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
------	--------	-------	------------	---------------	-----------	--------------

REGISTERED CIVIL ENGINEER
No. _____
PLANS APPROVAL DATE _____
CIVIL STATE OF CALIFORNIA
PARSONS BRINCKERHOFF QUADE & DOUGLAS, INC.
2485 NATOMAS PARK DR. STE. 400
SACRAMENTO, CA 95833
The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.



FOOTING PLAN

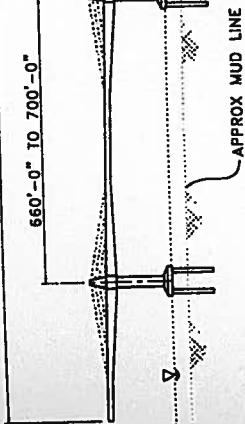


300' & 550' SPANS

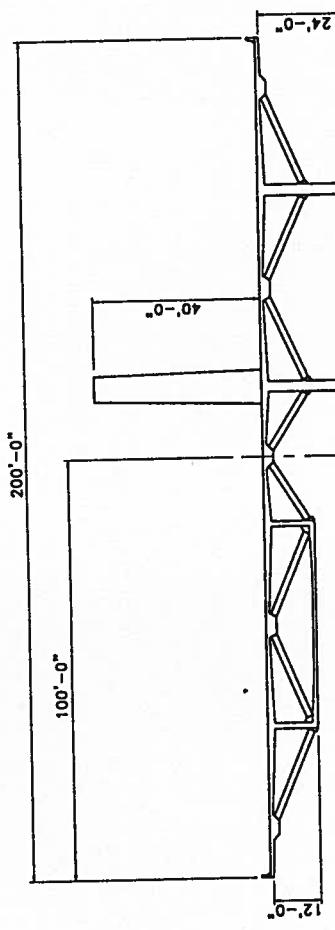
PRELIMINARY DESIGN

DESIGN DETAILS		PREPARED FOR THE STATE OF CALIFORNIA		VIADUCT SUPERSTRUCTURE ELEVATION	
QUANTITIES	BY	BY	PROJECT ENGINEER	BY	REVISION DATES (PRELIMINARY STAGE ONLY)
DISCARD PRINTS BEARING EARLIER REVISION					
STOR OFF DATE					

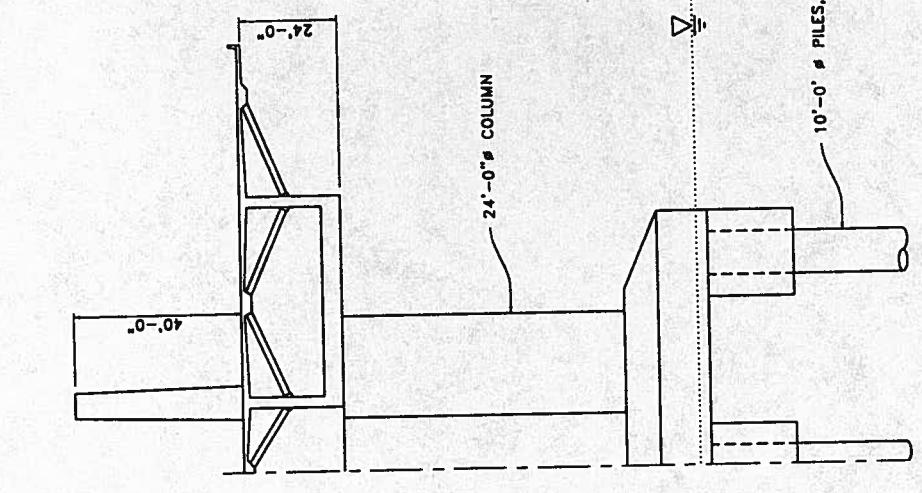
LENGTH OF APPROACH LINK VARIES FROM 660'-0" TO 7000'-0" (VARIES FROM 10 SPANS @ 660'-0" TO 10 SPANS @ 700'-0")



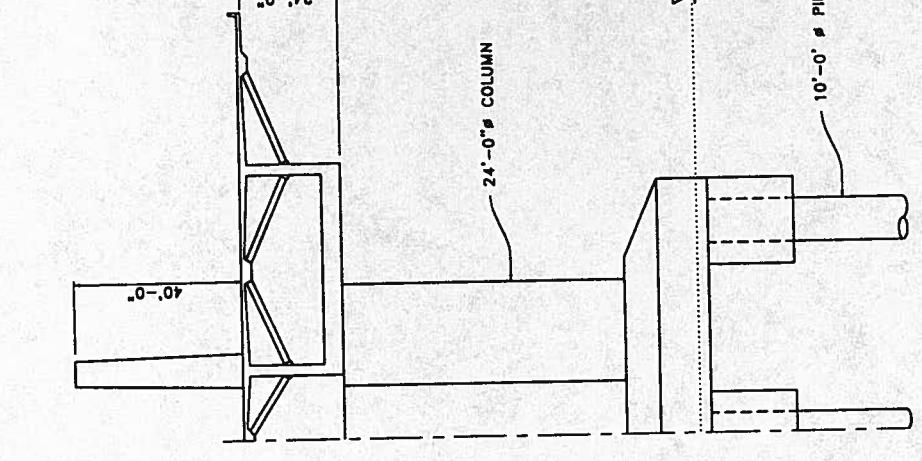
ELEVATION
SCALE: 1:200



NEAR MIDSPAN



NEAR PIER OPTION - 1



NEAR PIER OPTION - 2

NOTE:
ARTICULATION OF SUPERSTRUCTURE
TO SUBSTRUCTURE TO BE FIXED OR
ISOLATED.

SFOBB
EAST BAY CROSSING
REPLACEMENT

EXTRADOSSED APPROACH LINK
ALTERNATIVE

PB /HNTB

TYPICAL SECTION
SCALE: 1:20